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STABILITY AND STRESS ANALYSES MARSEILLES DAM, ILLINOIS WATERWAY

by

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20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

The monoliths of Marseilles Dam were analyzed to see if they meet present-day stability requirements.

In the analysis of the tainter gate monoliths, the keys connecting the pier and spillway sections were determined to be overstressed; therefore, the spillway and pier sections were considered to act independently.

Using this assumption, the pier and spillway sections were analyzed independently for stability considering the following load cases: (1)

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20. ABSTRACT (Continued)

a. Normal operation; (2)
b. Normal operation with ice; (3)
c. Normal operation with earthquake; and (4)
d. Flood condition.

The results showed that the spillway sections were adequate in stability, and the pier sections were inadequate against overturning. To correct the inadequacy of the pier sections, it was recommended that each pier section be posttensioned using a 602-kip force. The pier sections were reanalyzed to include the recommended posttensioning and were determined to be adequate in stability. The resulting stresses in the structure, foundation, and grouted anchors were computed and determined to be within allowables.

A previous stability investigation by the U. S. Army Engineer District, Chicago, concluded that the stability of the ice chute monoliths was adequate. This paper concurs with that conclusion.

PREFACE

The stability analysis of Marseilles Dam was performed in 1979 for the U. S. Army Engineer District, Chicago, by the Structures Laboratory (SL) of the U. S. Army Engineer Waterways Experiment Station (WES).

The contract was monitored by Messrs. Ignas Juzenas and George Sanborn. Their interest and help was greatly appreciated.

The study was performed under the direction of Messrs. B. Mather, W. J. Flathau, and J. M. Scanlon, SL. The structural analysis was performed by Dr. C. E. Pace, Messrs. R. L. Campbell and E. F. O'Neil, and SP5 John Z. Oak. The material properties were obtained by Mr. R. L. Stowe and WES Soils and Pavements Laboratory. The report was prepared by Dr. Pace and Mr. Campbell.

The Commanders and Directors of WES during the conduct of this test program and the preparation and publication of this report were COL John L. Cannon, CE, and COL Nelson P. Conover, CE. Mr. F. R. Brown was Technical Director.

CONTENTS

	Page
PREFACE	1
CONVERSION FACTORS, INCH-POUND TO METRIC (SI) UNITS OF MEASUREMENT	3
PART I: INTRODUCTION	4
Background	4
Stability Analysis	4
Stress Analysis	6
Objective	7
PART II: STABILITY ANALYSIS	8
Tainter Gate Monolith	8
Ice Chute Monolith	12
PART III: STRESS ANALYSIS	13
Pier	13
Pier Foundation	14
REFERENCES	17
APPENDIX A: STABILITY AND STRESS ANALYSIS DATA	

CONVERSION FACTORS, INCH-POUND TO METRIC (SI) UNITS OF MEASUREMENT

Inch-pound units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	by	To Obtain
feet	0.3048	metres
inches	0.0254	metres
kips (force) per square foot	47.88026	kilopascals
miles (U. S. statute)	1.6093	kilometres
pounds (force)	4.448222	newtons
pounds (mass)	0.4535924	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic metre
pounds (force) per square inch	0.006894757	megapascals
tons (force) per square foot	0.009576052	megapascals

STABILITY AND STRESS ANALYSES, MARSEILLES DAM, ILLINOIS WATERWAY

PART I: INTRODUCTION

Background

1. The Marseilles Dam is on the Illinois Waterway near Marseilles, Ill., which is about 60 miles* southeast of Chicago. Previously published reports by the U. S. Army Engineer District, Chicago (1973a, b, and c), present the overall view and sections of the dam. The material properties of the concrete and foundation are described by Stowe (1979).

2. Even though the Marseilles structures have been in service for a long time, it is important that they be examined to view their present condition in relation to present-day criteria to assure continued structural adequacy. If the design of the structure is judged to be inadequate or if the deterioration of the structure causes inadequacies, feasible modifications must be made.

Stability Analysis

3. One of the main considerations for structural adequacy of a dam is the stability of its various monoliths when subjected to possible loading conditions. Stability studies involve the analyses of selected monoliths to determine if they have adequate resistance against overturning, sliding, and base pressures.

Overturning

4. The adequacy of the structure to resist overturning can be judged by the location of the resultant with respect to the base of the

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* A table of factors for converting inch-pound units of measurement to metric (SI) units is presented on page 3.

section where stability is being considered within the dam, at the base-foundation interface, or at a plane or combination of planes below the base. In general, the gravity sections where stability against overturning is being considered are required to have the resultant of applied loads fall within the kern of the base of the section being analyzed. However, for operating conditions with earthquake, the resultant may fall outside of the kern, but within the base, as long as allowable foundation stresses are not exceeded.

5. The percent effective base (percent of the base which is in compression) is a good way of representing where the resultant falls in a rectangular based section. It is a good guide for representing overturning resistance for any shape base. An example for a rectangular base follows:

Percent Effective Base	Resultant Location Within Base
100	Within middle third or in kern area
75	At a quarter point of base
50	At a sixth point of base

Sliding

6. Sliding resistance of a monolith is calculated by choosing a trial failure plane or combination of planes and calculating the resistance along that path. The resistance may be composed of several types. The sliding resistance due to friction and cohesion of the surface between the monolith and its foundation is calculated by the shear-friction formula given in ETL 1110-2-184 (Department of the Army, Office, Chief of Engineers, 1974). However, the formula in this ETL is inadequate for evaluating structural sliding on inclined planes. The sliding resistance due to all or any part of the failure plane extending through either the concrete monolith or the foundation is calculated from the shearing strength of the material acting over the length in which shearing occurs.

7. In general, a shear-friction safety factor of 4 is required for all conditions of loading where earthquake is not considered and is 2-2/3 for loading conditions considering earthquake. In discussions

with the Office, Chief of Engineers, it was concluded that the following safety factors for sliding would be adequate.

Condition	Minimum Value for Safety Factor
a. Use angle of internal friction corresponding to the shear resistance of precut concrete-on-rock, reliable strut sistance, no key resistance, and no cohesion.	1.5
b. Condition "a" for earthquake loading.	1.15
c. Use angle of internal friction associated with the shear resistance of concrete cast on foundation rock, plus key resistance, plus cohesion, and plus reliable strut resistance.	4
d. Condition "c" for earthquake loading.	2-2/3

From the above, the criteria using safety factors of 1.5 and 1.15 will be considered only if the criteria using safety factors of 4 and 2-2/3 are exceeded.

Base pressure

8. The water pressure used to assure adequate design against earthquake was obtained as described in EM 1110-2-2200 (Department of the Army, Office, Chief of Engineers, 1958) (Westergaard Theory).

9. The base pressures are the sum of the contact and uplift pressures on the concrete-foundation interface.

Stress Analysis

10. The results of a three-dimensional stress analysis are needed to determine if there is any overstress in the concrete monolith or foundation due to correcting overturning deficiency by posttensioning the monolith to the foundation.

Objective

11. The objective of this study was to analyze the monoliths of the Marseilles Dam to see if they meet present-day stability requirements. If present-day criteria were not met, corrective measures were to be recommended.

PART III: STABILITY ANALYSIS

Tainter Gate Monolith

Stress in keys between pier and spillway

12. The typical geometry of the tainter gate monolith of Marseilles Dam is shown in Figure A1. The construction of the tainter gate monolith is such that the pier extends down to the foundation and is only connected to the overflow section by concrete keys.

13. The tainter gate monolith pier and spillway were analyzed for stability for the following loadings:

- a. Normal operation.
- b. Normal operation with ice.
- c. Normal operation with earthquake.
- d. Flood condition.

14. The first consideration concerning the stability of the tainter gate monolith is whether or not the concrete keys allow a significant transfer of shears and moments between the pier and overflow section in order that they can be considered monolithic. If the keys are stressed above the allowable limits, they will have to be considered ineffective and the pier and overflow sections analyzed independently for adequacy in stability. The calculations for approximate shear stress in the keys are given in Figures A2 and A3.

15. To determine the shear stress in the keys, the following assumptions were made:

- a. Keys are not sheared.
- b. The pier and spillway act as a unit resulting in no differential settlement between them.
- c. The strains under the pier and spillway are equal at a common point.

Using these assumptions, the shear force (ΔV) and its moment arm about the center of gravity of the base were determined by setting the base pressures equal for common points between the pier and spillway. The shear force (ΔV) and its associated torque (ΔM) were transferred to the

centroid of the keys, and the average maximum shear stress in the outside key was calculated as that produced by direct shear, plus the shear created by the torsion.

16. The maximum average shear stress on the downstream key was calculated to be 708 psi for the normal operation and 955 psi for normal operation with ice.

17. The shear stress produced in the keys is also increased by the applied horizontal forces. The contribution of the shear stress due to the horizontal forces is not calculated because that contributed by the vertical forces is already excessive. An allowable shear stress of $1.1 \sqrt{f'_c} = 1.1 \sqrt{9998} = 110$ psi is used. At this point it is seen that the keys cannot be depended upon to cause the pier and spillway sections to act monolithic. The stability analysis must then be performed for the pier and spillway as if they act independently.

Stability analysis of pier

18. A summary of the stability analysis results of the pier is presented in Table A1.

19. The analysis of the stability of the pier alone is presented in Figures A4-A7. The adequacy of the stability of the pier alone will be determined by its sufficiency in resistance to overturning, sliding, and base pressures.

20. The first trial solution for overturning of the pier was to determine whether the total base is in compression under the given operating condition. These calculations are necessary to determine if some area of the base is not in compression, thereby causing full uplift to exist under the noncompressive area. The pier is inadequate in its resistance to overturning. The tainter gate piers have to be posttensioned to the foundations to meet present-day criteria against overturning. The general details of the posttensioning are presented in Figure A8. The posttensioning force needed is 602 kips per pier and is proposed to be accomplished by six posttensioning holes per pier located as shown in Figure A8. The design calculations for posttensioning are given in Figure A9. After the piers are posttensioned to the foundation, they will have adequate resistance against overturning.

21. For anchoring the posttensioning tendons, it is recommended that a grout having a three-day compressive strength of 5000 psi be used. To surround the tendons inside the concrete pier, a cement-based grout should be used to bond the tendon and protect it against corrosion. This grouting should be done only after there is negligible additional loss of prestress with time.

22. The resistance to sliding was evaluated in relation to the criteria presented in paragraph 7. There were several possible failure planes and conditions considered for adequacy of the structure against sliding.

23. The shear resistance was calculated for (a) a clayey seam, (b) an open bedding plane, and (c) precut, concrete-on-rock to determine which governed for sliding at or just below the concrete foundation interface. For all cases, the precut, concrete-on-rock governed the sliding resistance of the pier, as presented in Figure A10.

24. The strut resistance against sliding for both the concrete and the foundation was computed and compared to determine which offered the least resistance. For all loadings the foundation strut governed (see Figure A11).

25. Two approaches were used to evaluate sliding factors of safety for the pier. The first (lower bound value) used the sliding resistance as the precut, concrete-on-rock under the pier and apron plus the shear resistance of the foundation strut along an open bedding plane. In this case the shear resistance of the key was neglected. This approach required a factor of safety of 1.15 for normal operation with earthquake and 1.5 for the other load cases.

26. The second approach used the sliding resistance as concrete cast on foundation rock under the pier and apron, plus the shear resistance of the foundation strut along an open bedding plane, plus the shear resistance of the key. This required a factor of safety of 2-2/3 for normal operation plus earthquake and 4.0 for the other load cases.

27. The factor of safety against sliding for all loadings was adequate. There is significant scour at various locations along the toe of the stilling basin, therefore, maintenance needs to be performed

on this scour to eliminate it and assure that it does not continue to cause problems in the future. Using shear strengths of precut, concrete-on-rock, and strut action assuming an open bedding plane and no key resistance, the safety factors against sliding are 2.91 for normal operation, 2.28 for normal operation with earthquake, and 1.35 for normal operation with ice. The only condition of concern is when the strut action is ineffective for normal operation with ice, which produces a safety factor of 0.85. For this and to eliminate future maintenance problems, it is desirable to perform corrective maintenance to eliminate the scour at the downstream end of the stilling basin.

28. The bearing pressures were within allowable values

$$\left(\frac{\text{unconfined compressive strength}}{4} = 77.8 \text{ ksf}\right)$$

at the structure and foundation interface. There is a weaker stratum in the foundation at approximately 18 ft below the structure-foundation interface. The allowable foundation pressure

$$\left(\frac{UC}{4} = 15.48 \text{ ksf}\right)$$

is exceeded before the piers are prestressed to the foundation but is not exceeded for this weaker stratum after posttensioning. The calculations of foundation stresses at 18 ft below structure are presented in Figure A12.

Stability analysis of spillway

29. A summary of the stability analysis of the spillway is presented in Table A2. The detailed calculations of this analysis are presented in Figures A13-A16.

30. The resistance against overturning of the spillway is considered adequate, as the resultant for each loading falls within the kern area of the base resulting in 100 percent of the base being in compression.

31. The sliding resistance along a clayey seam and open bedding plane of the foundation was computed and compared to the residual-shear resistance of the concrete on rock. For all loadings the residual resistance governed. This comparison is presented in Figure A17. The sliding factor of safety was computed for both residual-shear resistance of concrete on rock and shear resistance of the natural joint between the concrete and the foundation rock. The resistance for each was compared with the allowables presented in paragraph 7 and thereby determined to be adequate. The shear resistance of the key and strut was not needed to determine adequacy against sliding and therefore was neglected in these calculations.

32. The bearing pressures at the spillway and foundation interface were well within the allowable. The allowable bearing pressure at the interface was determined to be 77.8 ksf using the unconfined compressive strength of 311 ksf from laboratory tests and a safety factor of 4. It was not necessary to check the bearing pressure at interface between the two different foundation materials as the bearing of the pier was greater than that of the spillway and it governed.

Ice Chute Monolith

33. Calculations and a discussion concluding that the ice chute stability was adequate were published by the Chicago District (1973b). These conclusions were verified; therefore, the stability of the ice chute is adequate.

PART III: STRESS ANALYSIS

Pier

Stress in pier at el 480.33 and 469*

34. Conventional stress analysis was used to determine the stresses in the concrete due to uniaxial and biaxial bending for normal operation with ice, before and after prestressing. Details of the stress analysis for sections at el 480.33 and 469 are presented in Figures A18-A21. In the biaxial analysis, bending is caused by one gate being out of the water while the other is still loaded.

35. The maximum stress values at these elevations are presented below. Before posttensioning, the piers are inadequate because of tensile stresses. After posttensioning, the piers do not have any tensile stress and are therefore adequate.

Elevation	Bending	Post-tension	Maximum Tensile Stress, psi	Maximum Compressive Stress, psi
480.33	Uniaxial	Before	-11.60	33.82
		After	None	26.60
480.33	Biaxial	Before	-16.32	42.71
		After	None	35.35
469.00	Uniaxial	Before	-21.32	62.99
		After	None	55.28
469.00	Biaxial	Before	-20.56	66.11
		After	None	53.82

Bearing stresses in pier directly beneath applied posttensioned force

36. Bearing stresses in the pier directly beneath the applied posttensioning force were limited to the allowable of $0.375f'_c$ through the posttensioned bearing plate design. Therefore, no overstress in

* All elevations (el) cited herein are in feet referred to mean sea level (msl).

bearing exists in the pier due to the applied posttensioning. For this location in the pier, the compressive strength of the concrete (f'_c) is 8767 psi with an allowable bearing stress of 3288 psi. This design is presented in Figure A9.

Pier Foundation

Finite-element stress program

37. Introduction. A finite-element structural analysis program (SAP V) was used to compute the stresses in the foundation due to post-tensioned loading. This program was designed and programmed to be an effective and efficient computer program for analyzing very large, complex three-dimensional structural systems with no loss of efficiency in the solution of small problems. Twelve structural element types were included to increase the usability and flexibility of the program.

38. The capacity of the program is controlled by an "A" array containing 10,000 double precision words of storage. The size of this array can be changed to increase the capacity of the program by increasing the value of "MTOT1" in a routine labeled SAP V of the program.

39. Input. Each node in the system is described by a location and a set of boundary conditions. The location is input as either cartesian (x, y, z) or cylindrical (r, z, θ) coordinates. The boundary conditions are defined by three translations and three rotations.

40. Each element in the system is described by a set of nodes and a material type. Other element input includes material properties such as Young's modulus of elasticity, weight density, coefficient of thermal expansion, Poisson's ratio, and shear modulus.

41. Undeformed or deformed finite-element grids can be obtained directly from SAP V; at present, capabilities do not exist to directly plot stress by SAP V. If stress plots are desired, a way to plot them must be devised and the maximum and minimum stresses will have to be calculated as well as plotted.

42. Structural loadings are input as nodal and element loads. The nodal loads are applied as forces and moments. The element loads include thermal, gravity, and hydrostatic loadings.

43. Output. The solution output includes displacements and rotations for each unrestrained node and normal and shearing stresses at selected points for each element. The output units are the same as the input units.

Finite-element grid

44. A two-dimensional axisymmetric finite-element analysis was used to compute the stresses in the foundation. In this analysis a 25-ft depth of foundation was used with the prestress force being applied at the key foundation interface. The foundation was subdivided into two foundation materials and a hole was added for a posttensioned grouted anchor. The depth of the hole was limited to 20 ft due to the poor bond strength of the lower foundation material. The finite-element grid for this analysis is presented in Figure A22.

45. It was later determined that the posttensioning would not be placed through the key. Therefore, it was assumed that the 6 ft of foundation between the base of the structure and the bottom of key was the same as material 1 directly beneath the key. This resulted in the depth of material 1 being increased from 12 ft to 18 ft. The 18-ft depth was used in computing the required bond length for anchoring the structure to the foundation. It was not necessary to perform the finite-element analysis again as the increase in depth of material 1 would increase the volume and thereby increase the overall strength of the foundation and reduce stresses.

46. The bond strength for lower foundation material is 31.4 psi and is approximately one-seventh of that for the top material. As the maximum capacity of the 20-ft hole in bond is 150 kips, the required bond strength for the 100.3-kip working force is adequate. The maximum posttensioned force calculations are presented in Figure A9.

Stresses in the grout
surrounding posttensioning cable

47. The posttensioned load for a single location was the only loading applied to the foundation for the finite-element analysis. The plotted stress results for an applied working force of 100.3 kips are presented in Figure A23. These data show the stresses in the grout to be 56-psi tension at the foundation surface and 23-psi tension at a depth of 5 ft.

48. After applying the overburden stresses due to bearing pressures and the overlap stresses from adjacent posttensioning, the net stresses in the grout were 46-psi tension at the foundation surface and 8-psi tension at a depth of 5 ft. At a depth of 10 ft the net stress in the grout was 4-psi compression. Even with a conservative compressive strength of 5000 psi and the allowable tensile stress of 0.01 f_c' = 50 psi, the allowable exceeds the above tensile values for the grout. Therefore, it can be concluded that no overstress in tension exists in the grout due to the applied posttensioning.

Stresses in the foundation

49. The stresses in the foundation due to the applied working force were also presented in Figure A23. For posttensioning loads of a 100.3-kip working force and a 148.9-kip maximum temporary force, overburden and overlap stresses were included in the calculations for the net stresses at 3.33 in. from the applied loading. This is presented in Table A3 and Figure A24.

50. The 100.3-kip working force resulted in the only tensile stress in the foundation. This net tensile stress is only 0.1 psi and is located at the foundation surface 3.33 in. from the applied load. As this stress is negligible, no overstress in tension exists in the foundation due to the applied posttensioning.

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APPENDIX A: STABILITY AND STRESS ANALYSIS DATA

Table A1

Summary of Stability Analysis, Tainter Gate Pier

TAINTER GATE MONOLITH PIER	EFFECTIVE BASE (%)		SLIDING SAFETY FACTOR		FOUNDATION PRESSURE (KSF)*	
CASE LOADINGS	MINIMUM ALLOWABLE	CALCULATED	MINIMUM ALLOWABLE	CALCULATED	MAXIMUM ALLOWABLE	CALCULATED
BEFORE PRESTRESSING						
NORMAL OPERATION	100	92	1.50 4.00	2.91 12.77	78	8
NORMAL OPERATION WITH ICE	100	37	1.50 4.00	1.35 5.96	78	18
FLOOD CONDITION	100	100	1.50 4.00	4.23 18.75	78	7
NORMAL OPERATION WITH EARTHQUAKE	0	85	1.15 2.67	2.28 10.06	78	9
AFTER PRESTRESSING						
NORMAL OPERATION	100	100	1.50 4.00	3.67 15.92	78	7
NORMAL OPERATION WITH ICE	100	100	1.50 4.00	1.73 7.53	78	10
FLOOD CONDITION	100	100	1.50 4.00	5.43 23.62	78	7
NORMAL OPERATION WITH EARTHQUAKE	0	100	1.15 2.67	2.89 12.55	78	8

* FOUNDATION PRESSURE = INTERGRANULAR + UPLIFT

Table A2

Summary of Stability Analysis, Tainter Gate Spillway

TAINTER GATE MONOLITH SPILLWAY	EFFECTIVE BASE (%)		SLIDING SAFETY FACTOR		FOUNDATION PRESSURE (KSF)*	
CASE LOADINGS	MINIMUM ALLOWABLE	CALCULATED	MINIMUM ALLOWABLE	CALCULATED	MAXIMUM ALLOWABLE	CALCULATED
NORMAL OPERATION	100	100	1.50 4.00	1.75 19.42	78	3
NORMAL OPERATION WITH ICE	100	100	1.50 4.00	1.80 19.62	78	3
FLOOD CONDITION	100	100	1.50 4.00	2.40 27.87	78	2
NORMAL OPERATION WITH EARTHQUAKE	0	100	1.15 2.67	1.43 15.83	78	3

* FOUNDATION PRESSURE = INTERGRANULAR + UPLIFT

TABLE A3

VERTICAL COMPONENT OF STRESS (3.33" From Applied Posttensioning)
TAINTER GATE MONOLITH PIER - MARSEILLES L&D

FOUNDATION DEPTH (BELOW EL 459)	STRESS DUE TO ONE POSTTENSION HOLE		STRESS DUE TO ADJACENT HOLES		INTERGRANULAR STRESS DUE TO OVERBURDEN		NET STRESS	
	P=100.3k	P=148.9k	P=100.3k	P=148.9k	P=100.3k	P=148.9k	P=100.3k	P=148.9k
FT	PSI	PSI	PSI	PSI	PSI	PSI	PSI	PSI
0	-10.43	-15.49	- 1.27	- 1.89	11.60	25.00	- 0.10	7.62
1	- 7.77	-11.53	- 2.58	- 3.83	12.70	26.10	2.35	10.74
2	- 5.27	- 7.82	- 2.85	- 4.23	13.80	27.20	5.68	15.15
3	- 3.99	- 5.92	- 2.73	- 4.05	14.90	28.30	8.18	18.33
4	- 3.07	- 4.56	- 2.48	- 3.68	16.00	29.40	10.45	21.16
5	- 2.39	- 3.56	- 2.16	- 3.21	17.10	30.50	12.55	23.73
6	- 1.84	- 2.73	- 1.83	- 2.72	18.20	31.60	14.53	26.15
7	- 1.35	- 1.99	- 1.49	- 2.21	19.30	32.70	16.46	28.50
8	- 0.88	- 1.31	- 1.14	- 1.69	20.40	33.80	18.38	30.80
9	- 0.37	- 0.55	- 0.82	- 1.22	21.50	34.90	20.31	33.13
10	0.32	0.47	- 0.53	- 0.79	22.60	36.00	22.39	35.68
11	- 0.05	- 0.08	- 0.74	- 1.10	23.70	37.10	22.91	35.92
12	- 0.23	- 0.34	- 0.68	- 1.01	24.80	38.20	23.89	36.85
13	- 0.25	- 0.37	- 0.67	- 0.99	25.83	39.23	24.91	37.87
14	- 0.27	- 0.39	- 0.68	- 1.01	26.86	40.26	25.91	38.86
15	- 0.27	- 0.39	- 0.70	- 1.04	27.89	41.29	26.92	39.86
16	- 0.27	- 0.39	- 0.75	- 1.11	28.92	42.32	27.90	40.82
17	- 0.26	- 0.38	- 0.82	- 1.22	29.95	43.35	28.87	41.75
18	- 0.25	- 0.37	- 0.98	- 1.45	30.98	44.38	29.75	42.56
19	- 0.17	- 0.26	- 1.22	- 1.81	32.01	45.41	30.62	43.34
20	- 2.58	- 3.83	- 1.52	- 2.26	33.04	46.44	28.94	40.35
21	- 1.44	- 2.14	- 1.69	- 2.51	34.07	47.47	30.94	42.82
22	- 1.14	- 1.69	- 1.66	- 2.46	35.10	48.50	32.30	44.35
23	- 1.02	- 1.51	- 1.61	- 2.39	36.13	49.53	33.50	45.63
24	- 0.97	- 1.43	- 1.54	- 2.29	37.16	50.56	34.65	46.84

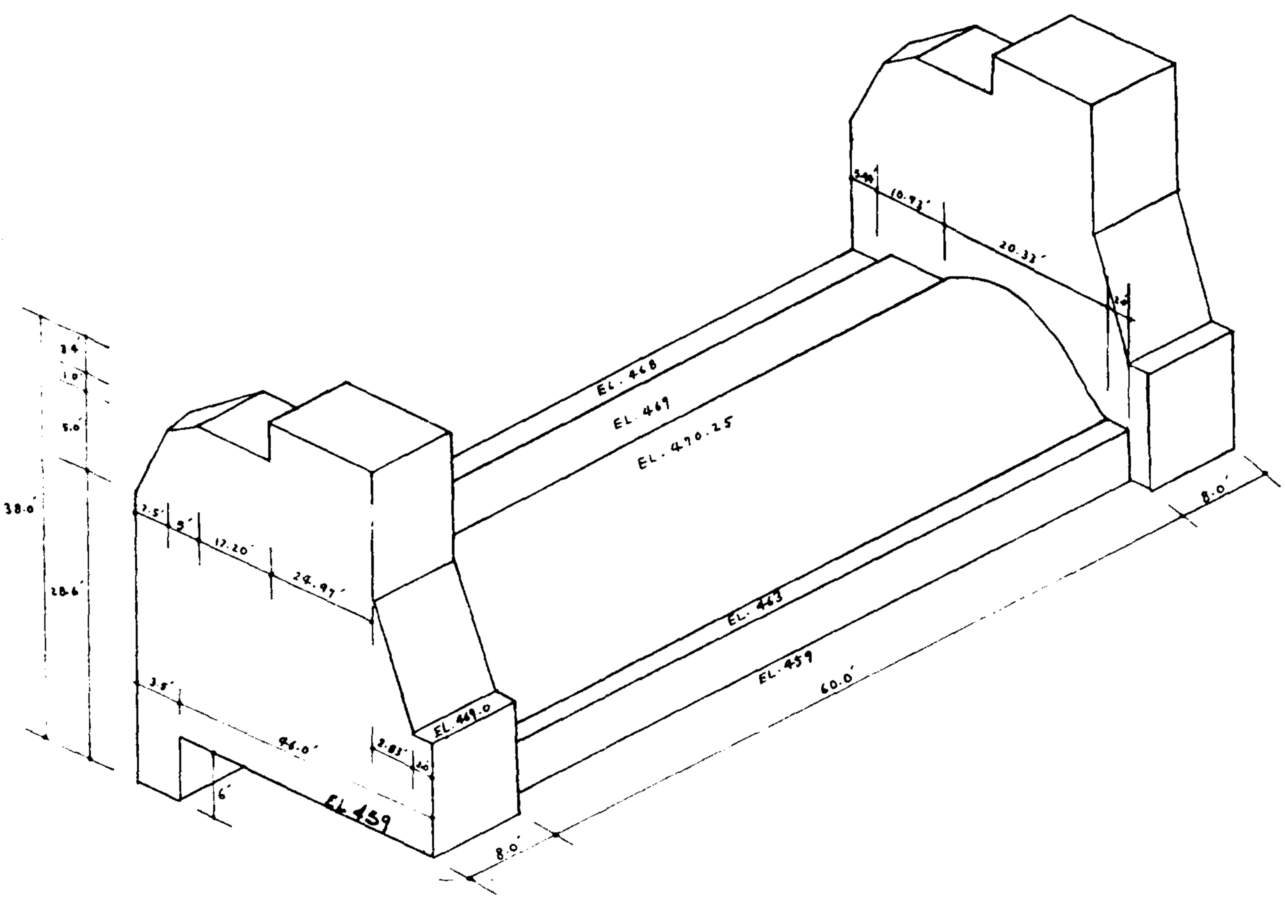
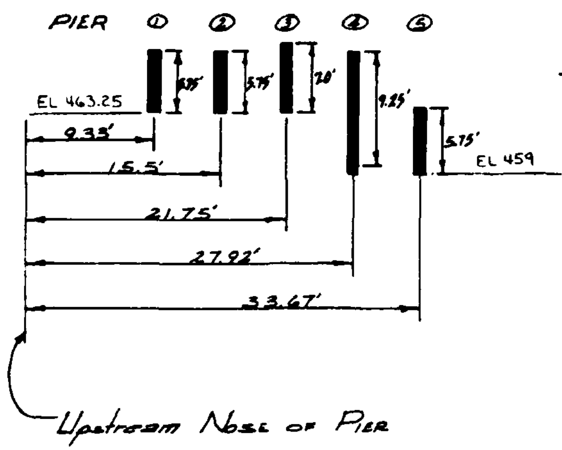


Figure A1. Typical tainter gate section (spillway and adjoining piers)



KEY GROUP CONFIGURATION

KEY	WIDTH	HEIGHT	AREA	Distance From Nose Of Pier To ℄ Of Key	Moment of Key Area About Upstream End of Pier	I_{xx}	I_{yy}	d	Ad^2
1	1	5.75	5.75	9.33	53.65	15.84	0.48	13.03	976.44
2	1	5.75	5.75	15.5	89.13	15.84	0.48	6.92	275.35
3	1	7.0	7.0	21.75	152.25	28.58	0.58	1.99	27.72
4	1	9.25	9.25	27.92	258.26	65.75	0.77	5.75	305.83
5	1	5.75	5.75	33.67	193.60	15.84	0.48	11.95	821.11
			33.5		746.89	142.05	2.79		2406.25

Horizontal Distance From Nose of Pier to Centroid of Key Areas $= \frac{746.89}{33.5} = 22.3'$

Polar Moment of Inertia $= \sum I_{xx} + \sum I_{yy} + \sum Ad^2$

$= 142.05 + 2.79 + 2406.25 \approx 2551\ ft^4$

Figure A2. Shear stress in keys between pier and spillway, tainter gate monolith, normal operation (Sheet 1 of 6)

LET:

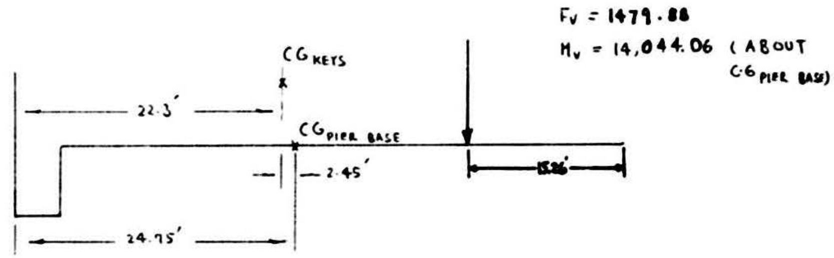
ΔV = SHEAR IN KEYS (KIPS)

ARM = MOMENT ARM OF ΔV ABOUT $CG_{PIER\ BASE}$ (FT)

ARM + 2.45' = ECCENTRICITY OF ΔV ABOUT CG_{KEYS} (FT)

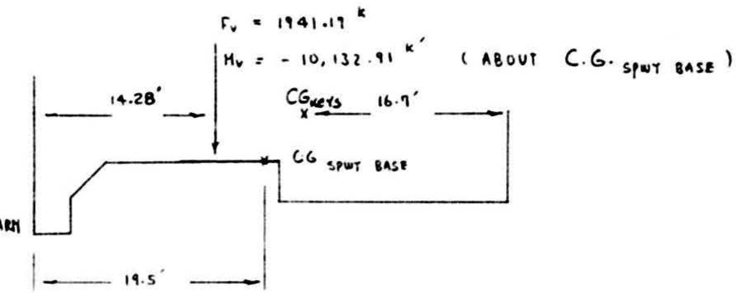
ΔM = TORQUE TRANSFERRED BY KEYS (FT-KIPS)

$\Delta M = \Delta V * [\text{ARM} + 2.45']$



PIER

BASE PRESSURES $\left(P = \frac{P}{A} \pm \frac{MC}{I} \right)$



PIER HEEL

$$P_{P\text{-}HEEL} = \frac{1479.88 - \Delta V}{8(49.5)} - \frac{[14{,}044.06 - \Delta V * \text{ARM}][24.75]}{(8)(49.5)^3/12}$$

$$P_{P\text{-}HEEL} = -0.5617 - 0.0025\,\Delta V + 0.00030609\,\Delta V * \text{ARM}$$

PIER AT 39' DOWNSTREAM OF HEEL

SPILLWAY

$$P_{P\text{-}39} = \frac{1479.88 - \Delta V}{8(49.5)} + \frac{(14.044.06 - \Delta V * \text{ARM})(14.25)}{(8)(49.5)^3/12}$$

$$P_{P\text{-}39} = 6.2121 - 0.002525\,\Delta V - 0.00017623\,\Delta V * \text{ARM}$$

Figure A2. (Sheet 2 of 6)

SPILLWAY HEEL

$$p_{S-HEEL} = \frac{1941.17 + \Delta V}{(60)(39)} - \frac{[-10132.91 + \Delta V * 5.25 + \Delta V * ARM][19.5]}{(60)(39)^3/12}$$

$$p_{S-HEEL} = 1.4958 + 0.00008218\ \Delta V - 0.00006575\ \Delta V * ARM$$

SPILLWAY TOE

$$p_{S-TOE} = \frac{1941.17 + \Delta V}{(60)(39)} + \frac{[-10132.91 + \Delta V * 5.25 + \Delta V * ARM][19.5]}{(60)(39)^3/12}$$

$$p_{S-TOE} = 0.1634 + 0.0007725\Delta V + 0.00006575\ \Delta V * ARM$$

SHEAR FORCE AND MOMENT IN KEYS

ASSUME KEYS do NOT shear and THE PIER and SPILLWAY ACT AS A UNIT resulting IN NO DIFFERENTIAL SETTLEMENT BETWEEN THEM. THIS WILL CAUSE THE STRAIN UNDER THE PIER AND SPILLWAY TO BE EQUAL AT A COMMON POINT between THEM.

FOR COMMON POINTS

$$\epsilon_{PIER} = \epsilon_{SPILLWAY}$$

$$E_{PIER} = E_{SPILLWAY}$$

Figure A2. (Sheet 3 of 6)

therefore, the stress at COMMON POINTS are EQUAL

$$p_{PIER} = \epsilon_{PIER} \, E_{PIER} \qquad (1)$$

$$p_{SPILLWAY} = \epsilon_{SPILLWAY} \, E_{SPILLWAY} \qquad (2)$$

SINCE the right side of EQUATIONS (1) and (2) are equal

$$p_{PIER} = p_{SPILLWAY}$$

for COMMON POINTS.

THEREfore,

$$p_{P\text{-}HEEL} = p_{S\text{-}HEEL}$$

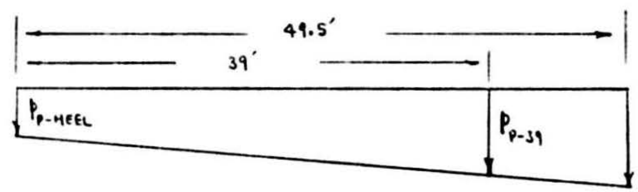
$$0.00037184 \, \Delta V * ARM - 0.002607 \, \Delta V = 2.0575$$

$$\text{(A)} \qquad \Delta V * ARM - 7.0111 \, \Delta V = 5533.29$$

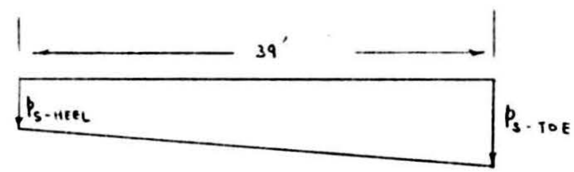
$$p_{P\text{-}39} = p_{S\text{-}TOE}$$

$$0.00024198 \, \Delta V * ARM + 0.003299 \, \Delta V = 6.0487$$

$$\text{(B)} \qquad \Delta V * ARM + 13.6251 \, \Delta V = 24{,}996.69$$



BASE PRESSURE DIAGRAM - PIER



BASE PRESSURE DIAGRAM - SPILLWAY

Figure A2. (Sheet 4 of 6)

SHEAR FORCE AND MOMENT IN KEYS (CONTINUED)

(A) - (B) $\qquad \Delta V = 943.17$ KIPS

$ARM = 12.88$ FT

$\Delta M = \Delta V * [ARM + 2.45']$

$\Delta M = 943.17 [12.88 + 2.45] = 14,459$ KIP - FT

CHECK TO SEE IF RESULTANT FALLS WITHIN MIDDLE THIRD OF BASE

PIER

$$R = \frac{\Sigma M}{\Sigma F} = \frac{4,044.06 - [943.17] * [12.89]}{1479.88 - 943.17}$$

$$R = 3.53 \text{ FT} < \frac{16.5}{2} \qquad \text{OK}$$

SPILLWAY

$$R = \frac{-10,132.91 + [943.17] * [12.88] + [943.17] * [5.25]}{1941.17 + 943.17}$$

$$R = 2.42 \text{ FT} < \frac{13.0}{2} \qquad \text{OK}$$

Figure A2. (Sheet 5 of 6)

SHEAR STRESS

C = MAXIMUM DISTANCE TO CENTROID OF ANY KEY (FT)

J = POLAR MOMENT OF INERTIA OF KEYS (FT^4)

τ = AVERAGE MAXIMUM SHEAR STRESS IN KEYS (PSI)

$$\tau = \frac{\Delta V}{A_{KEY}} + \frac{\Delta V * (ARM + 2.45) * C}{J}$$

$$\tau = \left[\frac{943.17}{33.5} + \frac{943.17\,(2.88 + 2.45)(3.03)}{2.551} \right] \left[\frac{1000}{144} \right]$$

$$\tau = 708 \text{ psi} > 1.1\sqrt{f'_c} = 1.1\sqrt{9998} = 110 \text{ psi}$$ (OVERSTRESSED)

Figure A2. (Sheet 6 of 6)

LET :

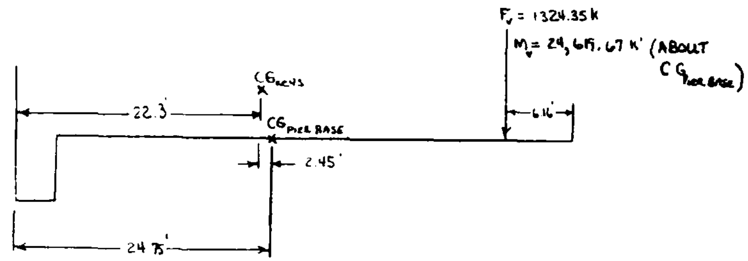
ΔV = SHEAR IN KEYS (KIPS)

ARM = MOMENT ARM OF ΔV ABOUT $CG_{PIER\ BASE}$ (FT)

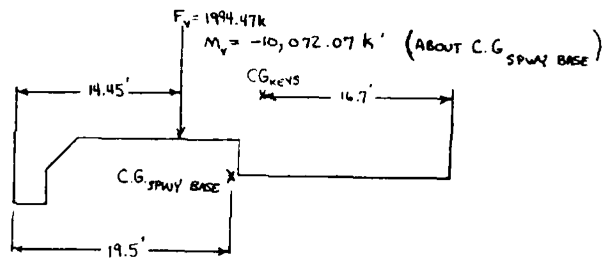
ARM + 2.45' = ECCENTRICITY OF ΔV ABOUT CG_{KEYS} (FT)

ΔM = TORQUE TRANSFERRED BY KEYS (FT-KIPS)

ΔM = $\Delta V * [ARM + 2.45']$



PIER



SPILLWAY

BASE PRESSURES $\left(p = \frac{P}{A} \pm \frac{Mc}{I} \right)$

PIER HEEL

$$p_{P\text{-}HEEL} = \frac{1324.35 - \Delta V}{8\ (49.5)} - \frac{[24619.67 - \Delta V * Arm]\ [24.75]}{(8)\ (49.5)^3/12}$$

$$p_{P\text{-}HEEL} = -4.1916 - 0.002525\ \Delta V + 0.00030609\ \Delta V * Arm$$

PIER AT 39' DOWNSTREAM OF HEEL

$$p_{P\text{-}39} = \frac{1324.35 - \Delta V}{8\ (49.5)} + \frac{[24619.67 - \Delta V * Arm]\ [14.25]}{(8)\ (49.5)^3/12}$$

$$p_{P\text{-}39} = 7.6831 - 0.002525\ \Delta V - 0.00017623\ \Delta V * Arm$$

Figure A3. Shear stress in keys between pier and spillway, tainter gate monolith, normal operation plus ice (Sheet 1 of 5)

SPILLWAY HEEL

$$p_{S\text{-HEEL}} = \frac{1994.47 + \Delta V}{(60)(39)} - \frac{[-10072.07 + \Delta V * 5.25 + \Delta V * \text{ARM}][19.5]}{(60)(39)^3/12}$$

$$p_{S\text{-HEEL}} = 1.5145 + 0.000082\ \Delta V - 0.00006575\ \Delta V * \text{ARM}$$

SPILLWAY TOE

$$p_{S\text{-TOE}} = \frac{1994.47 + \Delta V}{(60)(39)} + \frac{[-10072.07 + \Delta V * 5.25 + \Delta V * \text{ARM}][19.5]}{(60)(39)^3/12}$$

$$p_{S\text{-TOE}} = 0.1901 + 0.0000772\ \Delta V + 0.00006575\ \Delta V * \text{ARM}$$

SHEAR FORCE AND MOMENT IN KEYS

Assume keys do not shear and the pier and spillway act as a unit resulting in no differential settlement between them. This will cause the strain under the pier and spillway to be equal at a common point between them.

For common points

$$\epsilon_{PIER} = \epsilon_{SPILLWAY}$$

$$E_{PIER} = E_{SPILLWAY}$$

Figure A3. (Sheet 2 of 5)

therefore, the stress at common points are equal

$$p_{PIER} = \epsilon_{PIER} E_{PIER} \quad (1)$$

$$p_{SPILLWAY} = \epsilon_{SPILLWAY} E_{SPILLWAY} \quad (2)$$

since the right side of equations (1) and (2) are equal

$$p_{PIER} = p_{SPILLWAY}$$

for common points.

Therefore,

$$p_{P-HEEL} = p_{S-HEEL}$$

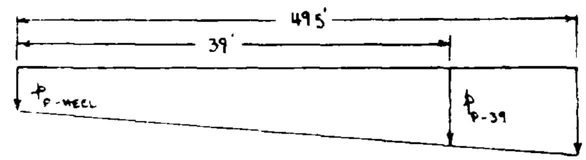
$0.00037184 \, \Delta V * Arm - 0.002607 \, \Delta V = 5.7061$

(A) $\Delta V * Arm - 7.0111 \, \Delta V = 15{,}345.58$

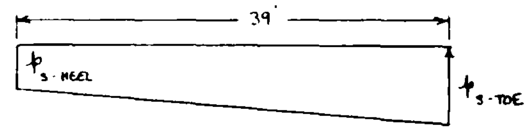
$$p_{P-39} = p_{S-TOE}$$

$0.00024198 \, \Delta V * Arm + 0.003297 \, \Delta V = 7.4930$

(B) $\Delta V * Arm + 13.6251 \, \Delta V = 30{,}965.37$



BASE PRESSURE DIAGRAM - PIER



BASE PRESSURE DIAGRAM-SPILLWAY

Figure A3. (Sheet 3 of 5)

SHEAR FORCE AND MOMENT IN KEYS (CONTINUED)

(A) - (B)

$\Delta V = 756.91$ Kips

ARM = 27.29 FT

$\Delta M = \Delta V * [\text{ARM} + 2.45']$

$\Delta M = 756.91 \text{ kips } [27.29 + 2.45] = 22{,}511 \text{ kip-FT}$

CHECK TO SEE IF RESULTANT FALLS WITHIN MIDDLE THIRD OF BASE

PIER

$$R = \frac{\Sigma M}{\Sigma F} = \frac{24{,}619.67 - [756.91] * [27.29]}{1324.35 - 756.91}$$

$$R = 6.99 \text{ FT} < \frac{16.5}{2} \quad \underline{\underline{OK}}$$

SPILLWAY

$$R = \frac{-10{,}072.07 + [756.91] * [27.29] + [756.91] * [5.25]}{1994.47 + 756.91}$$

$$R = 5.29 \text{ FT} < \frac{13.0}{2} \quad \underline{\underline{OK}}$$

Figure A3. (Sheet 4 of 5)

SHEAR STRESS

C = MAXIMUM DISTANCE TO CENTROID OF ANY KEY (FT)

J = POLAR MOMENT OF INERTIA OF KEYS (FT^4)

τ = AVERAGE MAXIMUM SHEAR STRESS IN KEYS (PSI)

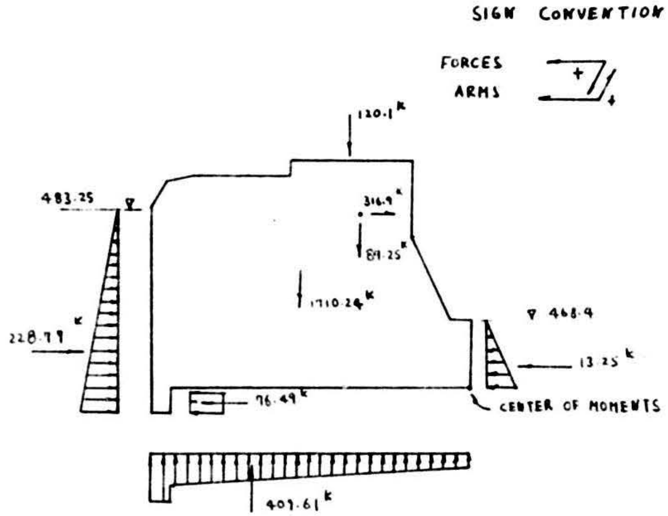
$$\tau = \frac{\Delta V}{A_{KEY}} + \frac{\Delta V * [ARM + 2.45] * C}{J}$$

$$\tau = \left[\frac{756.91}{33.5} + \frac{756.91\,[27.29+2.45](13.03)}{2551}\right]\left[\frac{1000}{144}\right]$$

τ = 9554 psi $> 1.1\sqrt{f'_c} = 1.1\sqrt{9998}$ = 110 psi (OVERSTRESSED)

Figure A3. (Sheet 5 of 5)

STABILITY ANALYSIS OF PIER ALONE



ITEM	FACTORS	F_V	F_H	ARM	MOMENTS
WT PIER	[.1552][{(10)(4.83) + (1/2)(2.83)(11.33) + (24.97)(32) + (28.6)(6.2) + (15)(3.5) + (9)(10) + (1)(7.56) + (1/2)(0.28)(3.4) + (5.94)(1)} (8) + (3.68)(0.68)(2.5) + (2.76)(0.68)(2.5) + (1.84)(0.68)(2.5) + (6)(1)(6) + (2)(8.5)(10.25) + (2)(1/2)(1.25)(10.25) + (1/2)(6)(3.75)(12.6) + (1/2)(5)(2.5)(6) + (7.25)(6)(17.6) + (2.5)(12.6)(6) + (1.25)(3) + (3.75)(3) + (3)(5)(3.75)]	1710.24		24.76	42345.54
WT TAINTOR ON PIER		89.25		8.83	788.08
THRUST TAINTOR ON PIER			-316.9	26.25	-8318.63
P HEAD WTR ON PIER	[.0625][(483.25 - 453)^2 (1/2)][8]		-228.77	4.08	-933.38
P TAIL WTR ON PIER	[.0625][(468.4 - 459)^2 (1/2)(8)][.6]		13.25	3.13	41.47
P WTR ON KEY	[(1)(1.356)(6) + (1/2)(1.831 - 1.356)(6)][8]		76.49	-3.15	-240.94
UPLIFT ON PIER	[(1.831)(3.5) + (1/2)(1.890 - 1.831)(3.5) + (0.587)(46) + (1/2)(1.356 - 0.587)(46)][8]	-409.61		28.80	-11796.77
WT BRIDGE	58.9 + (0.1)(9)(68)	120.1		13.67	1641.77
		1509.98	-455.93		23527.14

FIRST TRIAL SOLUTION TO DETERMINE IF PART OF THE BASE IS NOT IN COMPRESSION. IF SO, THEN FULL UPLIFT MUST BE APPLIED UNDER NON-COMPRESSION AREAS.

RESULTANT ARM

$$\frac{23527.14}{1509.98} = 15.58$$

PERCENT ACTIVE BASE

$$\frac{(15.58)(3)(100)}{49.5} = 94.42\ \%$$

Figure A4. Stability analysis, tainter gate monolith pier, normal operation (Sheet 1 of 3)

APPLYING FULL UPLIFT UNDER NON-COMPRESSION AREAS OF BASE

OVERTURNING

RESULTANT ARM

$$\frac{22577.59}{1479.88} = 15.26'$$

PERCENT ACTIVE BASE

$$\frac{(15.26)(3)(100)}{49.5} = 92.48\%$$

SLIDING

$R = F_v \tan 30° + [\text{Shear \& Cohesion Resistance Under Apron} + \text{Strut Resistance}]$

$= 1479.88 \tan 30° + [451.68] = 1306.09^{k}$

$$\text{FACTOR OF SAFETY} = \frac{1306.09}{450.80} = 2.91 > 1.5 \quad \underline{\underline{\text{OK}}}$$

$R' = F_v \tan 67° + CA_{BASE} + \text{Key Resistance} + [\text{Shear \& Cohesion Resistance Under Apron} + \text{Strut Resistance}]$

$R' = 1479.88 \tan 67° + 2.12(8)(46) + 3.5(8)(1.1\sqrt{9998})\left(\frac{144}{1000}\right) + [1047.54]$

$R' = 5757.55^{k}$

$$\text{FACTOR OF SAFETY} = \frac{5757.55}{450.80} = 12.77 > 4 \quad \underline{\underline{\text{OK}}}$$

BASE PRESSURE BEFORE POSTTENSIONING

INTERGRANULAR PRESSURE

$\sigma_{max} = \frac{2}{3}\frac{P}{cb} = \frac{(2)(1479.88)}{(3)(15.26)(8)} = 8.08$ ksf @ HEEL OF PIER $< \sigma_{ALLOWABLE} = \frac{\text{Unconfined Compressive Strength}}{4} = \frac{311}{4} = 77.8$ Ksf $\underline{\underline{\text{OK}}}$

$\sigma_{min} = 0$ @ 3.72 FT FROM UPSTREAM FACE OF PIER

Figure A4. (Sheet 2 of 3)

BASE PRESSURES AFTER POSTTENSIONING

INTERGRANULAR PRESSURE

$R = F_v + 6P = 1479.88 + 6[100.3] = 2081.68$ KIPS

$M = F_V e_v + 3P[(49.5 - 8.5) + (49.5 - 12.)]$

$= 1479.88[15.26] + 3[100.3][78.5] = 46203.62$ FT - KIPS

$e = \frac{M}{R} = \frac{46203.62}{2081.68} = 22.20$ FT

$\sigma_{max} = \frac{R}{A} + \frac{Rec}{I}$

$= \frac{2081.68}{(49.5)(8)} + \frac{2081.68[49.5/2 - 22.20][49.5/2]}{[8][49.5]^3/12}$

$= 5.26 + 1.62 = 6.88$ KSF at toe

$\sigma_{min} = 5.26 - 1.62 = 3.64$ KSF at heel

UPLIFT PRESSURE

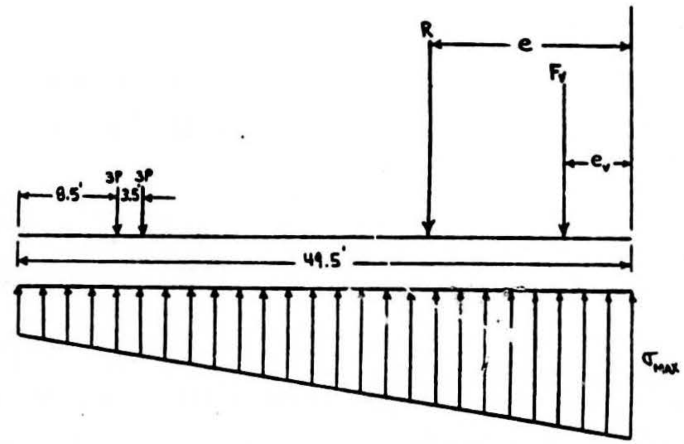
HEEL $(483.25 - 453)(.0625) = 1.89$ KSF

TOE $(468.4 - 459)(.0625) = 0.59$ KSF

TOTAL PRESSURE = INTERGRANULAR + UPLIFT

TOE $= 6.88 + 0.59 = 7.47$ KSF $< \sigma_{allow} = \frac{\text{Unconfined Compressive Strength}}{4} = 778$ KSF OK

HEEL $= 3.64 + 1.89 = 5.53$ KSF



SLIDING AFTER POSTTENSIONING

$R = F_v \tan 30° + [\text{Shear \& Cohesion Resistance Under Apron + Strut Resistance}]$
$= 2081.68 \tan 30° + 451.68$
$= 1653.54$

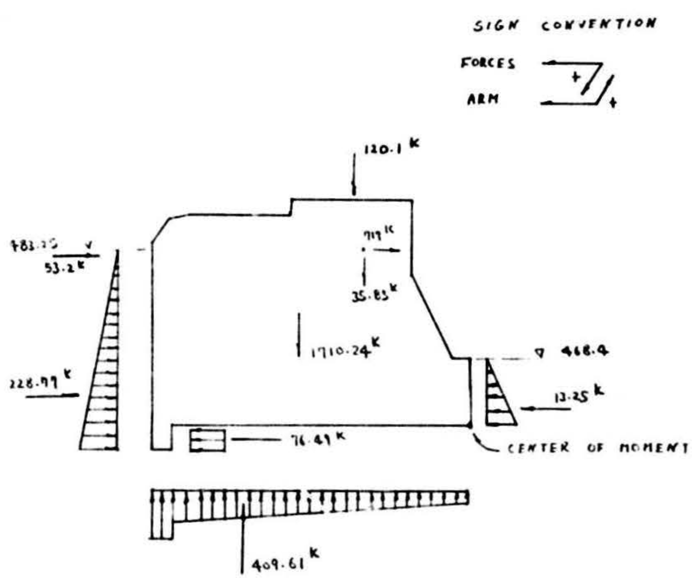
$SF = \frac{1653.54}{450.80} = 3.67 > 1.5$ OK

$R' = F_v \tan 67° + CA_{BASE} + \text{Key Resistance} + [\text{Shear \& Cohesion Resistance Under Apron + Strut Resistance}]$
$= 2081.68 \tan 67° + 2.12(8)(46) + (3.5)(8)(1.1)\sqrt{9998}\left(\frac{144}{1000}\right) + [1047.54]$
$= 7175.31$ Kips

$SF = \frac{7175.31}{450.80} = 15.92 > 4.0$ OK

Figure A4. (Sheet 3 of 3)

STABILITY ANALYSIS OF PIER ALONE



ITEM	FACTORS	Fv	FH	ARM	MOMENTS
WT PIER	[.1552][{(10)(4.83) + (1/2)(2.83)(11.33) + (24.99)(32) + (28.6)(6.2) + (15)(3.5) + (9)(10) + (1)(7.56) + (1/2)(0.28)(3.4) + (5.94)(1)}(8) + (3.68)(0.68)(2.5) + (2.76)(0.68)(2.5) + (1.84)(0.68)(2.5) + (6)(1)(6) + (2)(8.5)(10.25) + (2)(1/2)(1.25)(10.25) + (1/2)(6)(3.75)(12.6) + (1/2)(5)(2.5)(6) + (7.25)(6)(17.6) + (2.5)(12.6)(6) + (1.25)(3) + (3.75)(3) + (3)(5)(3.75)]	1710.24		24.76	42345.54
WT TAINTOR on PIER		35.83		8.83	316.53
THRUST TAINTOR on PIER			-717.0	26.23	-18821.25
THRUST OF ICE			-53.2	23.58	-1254.46
P HEAD WTR on PIER	[.0625][(483.25 - 453)² (1/2)][8]		-228.77	4.08	-933.33
P TAIL WTR on PIER	[.0625][(468.4 - 459)² (1/2)(8)][.6]		13.25	3.13	41.48
P WTR on KEY	[(1)(1.356)(6) + (1/2)(1.831 - 1.356)(6)][8]		76.49	-3.15	-240.94
UPLIFT on PIER	[(1.831)(3.5) + (1/2)(1.890 - 1.831)(3.5) + (0.587)(46) + (1/2)(1.356 - 0.587)(46)][8]	-409.61		28.80	-11796.77
WT BRIDGE	58.9 + (0.1)(9)(68)	120.10		13.67	1641.77
		1456.58	-909.23		11298.54

FIRST TRIAL SOLUTION TO DETERMINE IF PART OF THE BASE IS NOT IN COMPRESSION. IF SO, THEN FULL UPLIFT MUST BE APPLIED UNDER NON-COMPRESSION AREAS.

RESULTANT ARM

$$\frac{11298.54}{1456.58} = 7.76'$$

PERCENT ACTIVE BASE

$$\frac{(7.76)(3)(100)}{49.5} = 47.03\ \%$$

Figure A5. Stability analysis, tainter gate monolith pier, normal operation plus ice (Sheet 1 of 3)

APPLYING FULL UPLIFT UNDER NON-COMPRESSION AREAS OF BASE

OVERTURNING

RESULTANT ARM

$$\frac{8154.65}{1324.35} = 6.16 \text{ FT}$$

PERCENT ACTIVE BASE

$$\frac{(6.16)(3)(100)}{49.5} = 37.33\ \%$$

SLIDING

$R = F_v \tan 30° + [\text{Shear \& Cohesion Resistance Under Apron} + \text{Strut Resistance}]$

$= 1324.35 \tan 30° + [451.68]$

$= 1216.29$ Kips

$$\text{FACTOR OF SAFETY} = \frac{1216.29}{903.98} = 1.35$$

$R' = F_v \tan 67° + CA_{BASE} + \text{Key Resistance} + [\text{Shear \& Cohesion Resistance Under Apron} + \text{Strut Resistance}]$

$R' = 1324.35 \tan 67° + 2.12(8)(46) + 3.5(8)(1.1)\sqrt{9998}\left(\frac{144}{1000}\right) + [1047.54]$

$R' = 5391.15$ Kips

$$\text{FACTOR OF SAFETY} = \frac{5391.15}{903.98} = 5.96 < 4.00 \quad \underline{\underline{OK}}$$

BASE PRESSURE BEFORE POSTTENSIONING

INTERGRANULAR PRESSURE

$\sigma_{MAX} = \frac{2}{3}\frac{F_v}{C} = \frac{2(1324.35)}{3(6.16)(8)} = 17.92 \text{ ksf}$ @ HEEL OF PIER $< \sigma_{ALLOWABLE} = \frac{\text{Unconfined Compressive Strength}}{4} = \frac{311}{4} = 77.8 \text{ ksf}$ $\underline{\underline{OK}}$

$\sigma_{MIN} = 0$ @ 31.02' FROM UPSTREAM FACE OF PIER

Figure A5. (Sheet 2 of 3)

BASE PRESSURES AFTER POSTTENSIONING

INTERGRANULAR PRESSURE

$R = 1324.35 + 6[100.3] = 1926.15$ KIPS

$M = 1324.35[6.16] + 3[100.3][78.5] = 31,778.65$ FT-KIPS

$e = \frac{31,778.65}{1926.15} = 16.50$ FT

$\sigma_{max} = \frac{2}{3}\frac{R}{eb} = \frac{2}{3}\frac{1926.15}{(16.50)(8)} = 9.73$ KSF

$\sigma_{min} = 0.0$ KSF

UPLIFT PRESSURE

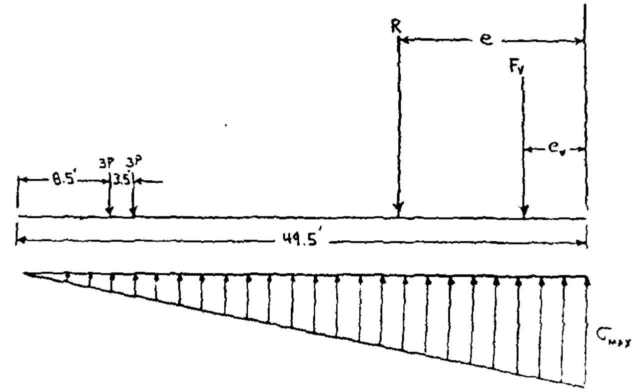
HEEL $(483.25 - 453)(.0625) = 1.89$ KSF

TOE $(468.4 - 459)(.0625) = 0.59$ KSF

TOTAL PRESSURE = INTERGRANULAR + UPLIFT

TOE $= 9.73 + 0.59 = 10.32 < \sigma_{Allow} = 77.8$

HEEL $= 0.0 + 1.89 = 1.89$



SLIDING AFTER POSTTENSIONING

$R = F_v \tan 30° +$ [Shear & Cohesion Resistance Under Apron + Strut Resistance]

$= 1926.15 \tan 30 + [451.68] = 1563.74$ kips

$SF = \frac{1563.74}{903.98} = 1.73 > 1.5$ OK

$R' = F_v \tan 67° + CA_{BASE}$ + Key Resistance + [Shear & Cohesion Under Apron + Strut Resistance]

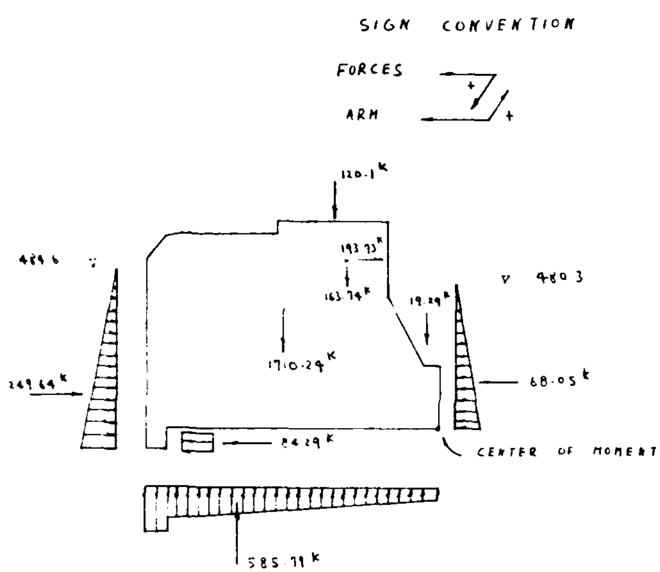
$= 1926.15 \tan 67° + 212(8)(44) + (3.5)(8)(1.1)\sqrt{4998}\left(\frac{144}{1000}\right) + 1047.54$

$= 6808.90$ kips

$SF = \frac{6808.90}{903.98} = 7.53 > 4.0$ OK

Figure A5. (Sheet 3 of 3)

STABILITY ANALYSIS OF PIER ALONE



ITEM	FACTORS	F_V	F_H	ARM	MOMENTS
WT PIER	[.1552][{(10)(4.83) + (½)(2.83)(11.33) + (24.99)(4) + (28.6)(6.2) + (15)(3.5) + (9)(10) + (1)(7.56) + (½)(0.28)(3.4) + (5.94)(1)}(8) + (3.68)(0.68)(2.5) + (2.76)(0.68)(2.5) + (1.84)(0.68)(2.5) + (6)(1)(6) + (2)(8.5)(10.25) + (2)(½)(1.25)(10.25) + (½)(6)(3.75)(12.6) + (½)(5)(2.5)(6) + (9.25)(6)(17.6) + (2.5)(12.6)(6) + (1.25)(3) + (3.75)(3) + (3)(5)(3.75)]	1710.24		24.76	42345.54
WT TAINTOR ON PIER		163.74		8.83	1445.82
THRUST TAINTOR ON PIER			-193.73	26.25	-5085.41
$P_{HEAD\ WTR}$ ON PIER	$[.0625][484.6 - 453]^2(½)(8)$		-249.64	4.53	-1130.87
$P_{TAIL\ WTR}$ ON PIER	$[.0625][480.3 - 459]^2(½)(8)(.6)$		68.05	7.1	483.16
P_{WTR} ON KEY	[(1.554)(6) + (½)(1.958 - 1.554)(6)](8)		84.29	-3.11	-262.14
WT TAILWTR ON PIER	[.0625][(½)(480.3 - 469)(2.83) + (480.3 - 469)(2)](8)	19.29		1.81	34.91
UPLIFT PIER	[(1.958)(3.5) + (½)(.019)(3.5) + (1.33)(2)(46) + (½)(.222)(46)][8]	-585.79		25.86	-15148.53
WT BRIDGE	58.9 + (0.1)(9)(68)	120.10		13.67	1641.77
		1427.58	-291.03		24324.25

FIRST TRIAL SOLUTION TO DETERMINE IF PART OF THE BASE IS NOT IN COMPRESSION. IF SO, THEN FULL UPLIFT MUST BE APPLIED UNDER NON-COMPRESSION AREAS.

RESULTANT ARM

$$\frac{24324.25}{1427.58} = 17.04$$

PERCENT ACTIVE BASE

$$\frac{(17.04)(3)(100)}{49.5} = 100\%$$

Figure A6. Stability analysis, tainter gate monolith pier, flood condition (Sheet 1 of 3)

SLIDING

$R = F_v \tan 30^\circ$ + Shear & Cohesion Resistance Under Apron + Strut Resistance]

$= 1427.58 \tan 30^\circ + [407.87]$

$= 1232.08$ kips

FACTOR OF SAFETY $= \dfrac{1232.08}{291.03} = 4.23 > 1.5$ OK

$R' = F_v \tan 67^\circ + CA_{BASE}$ + Key Resistance + [Shear & Cohesion Resistance Under Apron + Strut Resistance]

$R' = 1427.58 \tan 67^\circ + 2.12(8)(46) + 3.5(8)(1.1)\sqrt{4998}\left(\frac{144}{1000}\right) + [868.80]$

$R' = 5455.61$

FACTOR OF SAFETY $= \dfrac{5455.61}{291.03} = 18.75 > 4.00$ OK

BASE PRESSURE BEFORE POSTTENSIONING

INTERGRANULAR PRESSURE

$$\sigma = \frac{F_v}{A} \pm \frac{F_v e \cdot c}{I} = \frac{1427.58}{(49.5)(8)} \pm \frac{(1427.58)(24.75 - 17.04)(24.75)}{\frac{(8)(49.5)^3}{12}} = 3.61 \pm 3.37$$

$\sigma_{max} = 6.98$ ksf @ HEEL OF PIER $< \sigma_{ALLOWABLE} = \dfrac{\text{Unconfined Compressive Strength}}{4} = 77.8$ ksf OK

$\sigma_{min} = 0.24$ ksf @ TOE OF PIER

Figure A6. (Sheet 2 of 3)

BASE PRESSURES AFTER POSTTENSIONING

INTERGRANULAR PRESSURE

$R = F_V + 6P = 1427.58 + 6[100.3] = 2029.38$ KIPS

$M = F_V e_V + 3P[(49.5 - 8.5) + (49.5 - 12)]$

$= 1427.58[17.04] + 3[100.3][78.5] = 47946.61$ KIPS

$e = \frac{M}{R} = \frac{47946.61}{2029.38} = 23.63$ FT

$\sigma_{max} = \frac{R}{A} + \frac{Rec}{I}$

$= \frac{2029.38}{(49.5)(8)} + \frac{2029.38[49.5/2 - 23.63][49.5/2]}{[8][49.5]^3/12}$

$= 5.12 + 0.70 = 5.82$ KSF AT TOE

$\sigma_{min} = 5.12 - 0.70 = 4.42$ KSF AT HEEL

UPLIFT PRESSURE

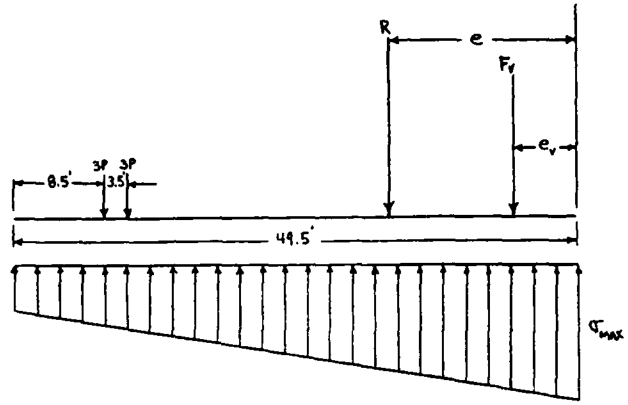
HEEL $(484.6 - 453)(.0625) = 1.98$ KSF

TOE $(480.3 - 459)(.0625) = 1.33$ KSF

TOTAL PRESSURE = INTERGRANULAR + UPLIFT

TOE $= 5.82 + 1.33 = 7.15$ KSF $< \sigma_{ALLOWABLE} = 77.8$ KSF OK

HEEL $= 4.42 + 1.98 = 6.40$ KSF



SLIDING AFTER POSTTENSIONING

$R = F_V \tan 30^\circ + $[Shear & Cohesion Resistance Under Apron + Strut Resistance]

$= 2029.38 \tan 30^\circ + [407.87]$
$= 1579.53$ Kips

$S.F. = \frac{1579.53}{291.03} = 5.43$ OK

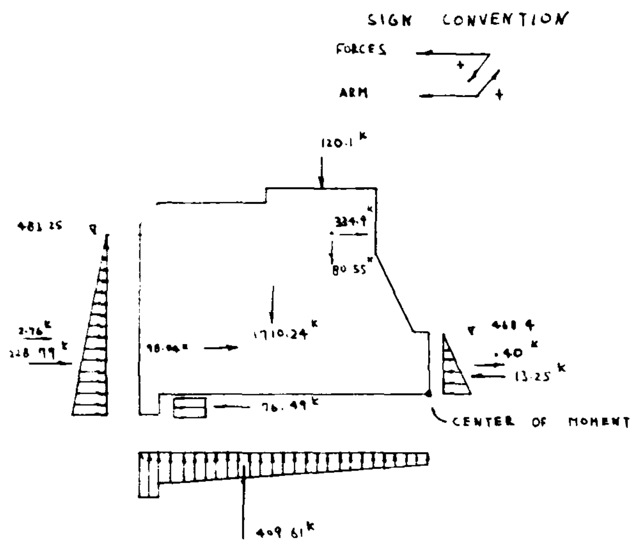
$R' = F_V \tan 67^\circ + CA_{BASE} + $ Key Resistance + [Shear & Cohesion Resistance Under Apron + Strut Resistance]

$= 2029.38 \tan 67^\circ + 2.12(8)(46) + 3.5(8)(1.1)\sqrt{9998}\left(\frac{144}{1000}\right) + [868.80]$
$= 6873.36$ Kips

$S.F. = \frac{6873.36}{291.03} = 23.62 > 4.0$ OK

Figure A6. (Sheet 3 of 3)

STABILITY ANALYSIS OF PIER ALONE



ITEM	FACTORS	F_V	F_H	ARM	MOMENT
WT PIER	[.1552][[(10)(4.83) + (½)(2.83)(11.23) + (24.97)(32) + (28.6)(6.2) + (15)(3.5) + (9)(10) + (1)(7.56) + (½)(0.28)(3.4) + (5.94)(1)](8) + [(3.68)(0.68)(2.5) + (2.76)(0.68)(2.5) + (1.84)(0.68)(2.5) + (6)(1)(6) + (2)(8.5)(10.25) + (2)(½)(1.25)(10.25) + (½)(6)(3.75)(12.6) + (½)(5)(2.5)(6) + (7.25)(6)(17.6) + (2.5)(17.6)(6) + (1.25)(3) + (3.75)(3) + (3)(5)(3.75)]	1710.24		24.76	42345.54
WT TAINTOR ON PIER		80.55		8.83	711.26
THRUST TAINTOR ON PIER			-334.9	26.25	-8791.13
P_{HEAD} WTR ON PIER	[.0625][(483.25 - 453)^2(½)][8]		-228.99	4.08	-933.38
P_{TAIL} WTR ON PIER	[.0625][(468.4 - 459)^2(½)(8)][.6]		13.25	3.13	41.47
P_{WATER} ON KEY	[(1)(1.356)(6) + (½)(1.831 - 1.356)(6)][8]		76.49	-3.15	-240.94
Uplift on PIER	[(1.831)(3.5) + (½)(1.890 - 1.831)(3.5) + (0.587)(46) + (½)(1.356 - 0.587)(46)][8]	-409.61		28.80	-11796.97
Pe.	0.05 [1710.2 + 83.6 + 167]		-98.04	14.27	-1399.03
Pe_2 HEAD	(2/3)(51)(0.05)(14.25)$\sqrt{14.25^2}$ (8)		-2.76	15.70	-43.33
Pe_2 TAIL	(2/3)(51)(0.05)(5.4)$\sqrt{5.4^2}$ (8)		-.40	6.17	-2.47
WT BRIDGE	58.9 + (0.1)(9)(68)	120.10		13.67	1641.77
		1501.28	-575.13		21532.99

FIRST TRIAL SOLUTION TO DETERMINE IF PART OF THE BASE IS NOT IN COMPRESSION. IF SO, THEN FULL UPLIFT MUST BE APPLIED UNDER NON-COMPRESSION AREAS

RESULTANT ARM

$$\frac{21532.99}{1501.28} = 14.34'$$

PERCENT ACTIVE BASE

$$\frac{(14.34)(3)(100)}{49.5} = 86.91\ \%$$

Figure A7. Stability analysis, tainter gate monolith pier, normal operation plus earthquake (Sheet 1 of 3)

APPLYING FULL UPLIFT UNDER NON-COMPRESSION AREAS OF BASE

OVERTUNING (Using Uplift from Normal Operation)

RESULTANT ARM

$$\frac{20{,}552.20}{1470.16} = 13.98'$$

PERCENT ACTIVE BASE

$$\frac{(13.98)(3)(100)}{49.5} = 84.72\ \%$$

SLIDING

$R = F_v \tan 30° + [\text{Shear \& Cohesion Resistance Under Apron + Strut Resistance}]$

$= 1470.16 \tan 30° + [451.68]$

$= 1300.48$ Kips

FACTOR OF SAFETY $= \frac{1300.48}{569.88} = 2.28 > 1.15$ OK

$R' = F_v \tan 67° + CA_{BASE} + \text{Key Resistance} + [\text{Shear \& Cohesion Resistance Under Apron + Strut Resistance}]$

$= 1470.16 \tan 67° + 2.12(8)(46) + 3.5(8)(1.1)\sqrt{4998}\left(\frac{144}{1000}\right) + [1047.54]$

$= 5734.66$ Kips

FACTOR OF SAFETY $= \frac{5734.66}{569.88} = 10.06 > 2.67$ OK

BASE PRESSURE BEFORE POSTTENSIONING

INTERGRANULAR PRESSURE

$\sigma_{max} = 2/3\ F_v/e$

$= \frac{(2)(1470.16)}{(3)(13.98)(8)} = 8.76$ ksf @ HEEL OF PIER $< \sigma_{ALLOWABLE} = \frac{\text{Unconfined Compressive Strength}}{4} = \frac{311}{4} = 77.8$ ksf OK

$\sigma_{min} = 0$ @ 8.01' FROM UPSTREAM FACE OF PIER

Figure A7. (Sheet 2 of 3)

BASE PRESSURE AFTER POSTTENSIONING

INTERGRANULAR PRESSURE

$R = 1470.16 + 6(100.3) = 2071.96$ KIPS

$M = 1470.16(13.98) + 3(100.3)(18.5) = 44,173.49$ FT-KIPS

$e = \frac{44,173.49}{2071.96} = 21.32$ FT

$\sigma_{max} = \frac{2071.96}{(49.5)(8)} + \frac{2071.96(49.5/2 - 21.32)(49.5/2)}{(8)(49.5)^3/12}$

$\sigma_{max} = 5.23 + 2.18 = 7.41$ KSF

$\sigma_{min} = 5.23 - 2.18 = 3.05$ KSF

UPLIFT PRESSURE

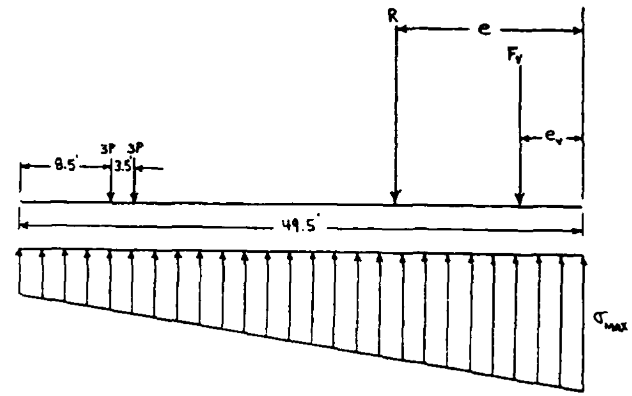
HEEL $(483.25 - 453)(.0625) = 1.89$ KSF

TOE $(468.4 - 459)(.0625) = .59$ KSF

TOTAL PRESSURE = INTERGRANULAR + UPLIFT

TOE $= 7.41 + 0.59 = 8.00$ KSF $< \sigma_{ALLOWABLE} = 77.8$ KSF OK

HEEL $= 3.05 + 1.89 = 4.94$ KSF



SLIDING AFTER POSTTENSIONING

$R = F_v \tan 30° + [$Shear & Cohesion Resistance Under Apron + Strut Resistance$]$

$= 2071.96 \tan 30° + [451.68]$

$= 1647.93$ Kips

$SF = \frac{1647.93}{569.88} = 2.89 > 1.15$ OK

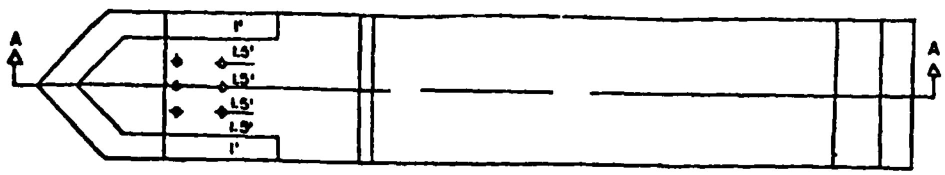
$R' = F_v \tan 67° + CA_{BASE} +$ Key Resistance $+ [$Shear & Cohesion Resistance Under Apron + Strut Resistance$]$

$= 2071.96 \tan 67° + 2.12(8)(46) + 3.5(8)(1.1)\sqrt{9998}\left(\frac{144}{1000}\right) + [1047.54]$

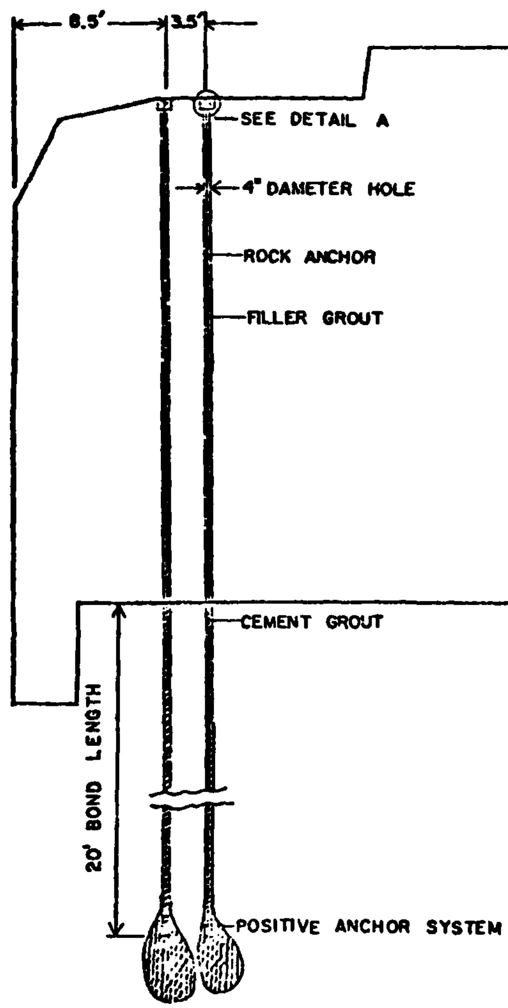
$= 7152.41$ Kips

$SF = \frac{7152.41}{569.88} = 12.55 > 2.67$ OK

Figure A7. (Sheet 3 of 3)

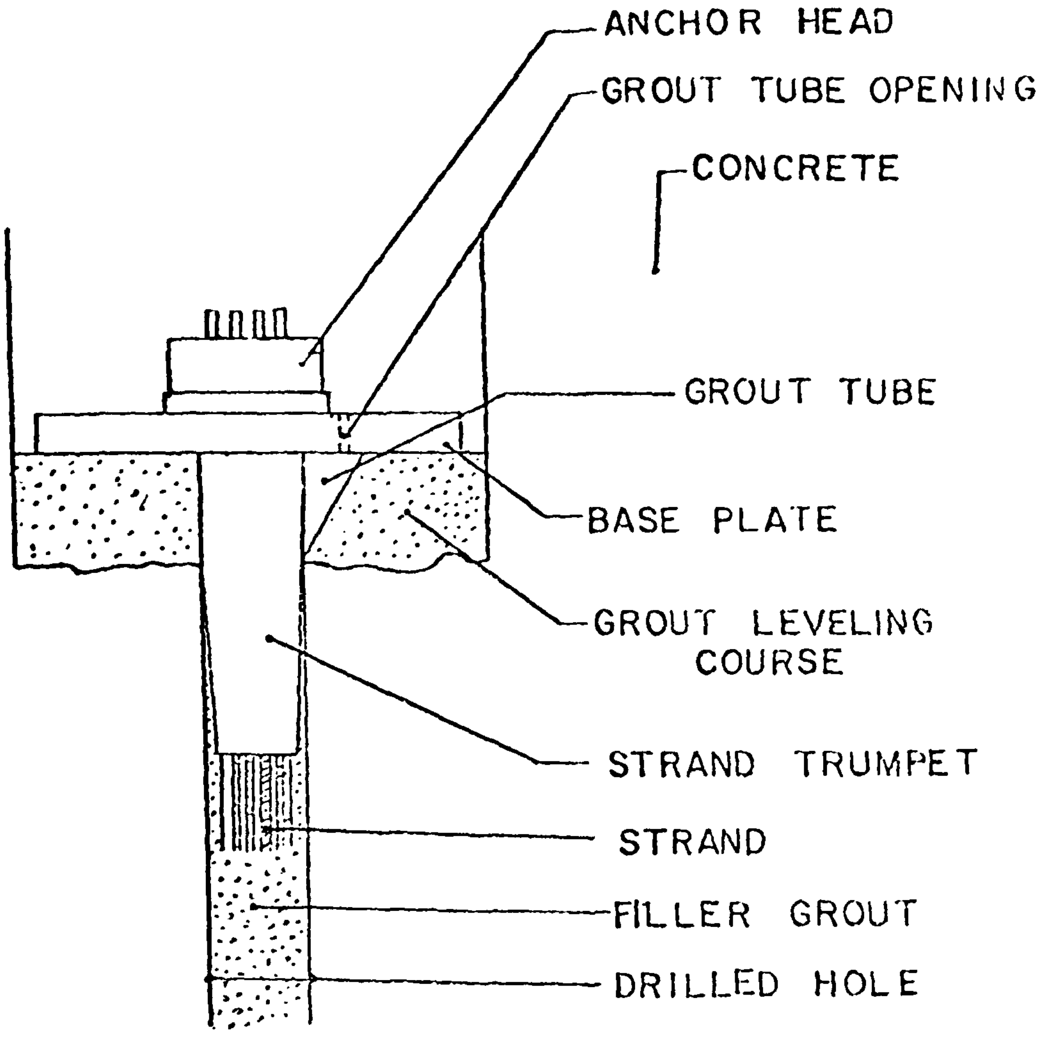


PLAN VIEW



SECTION A-A

Figure A8. Posttensioning design details, tainter gate monolith pier (Sheet 1 of 2)

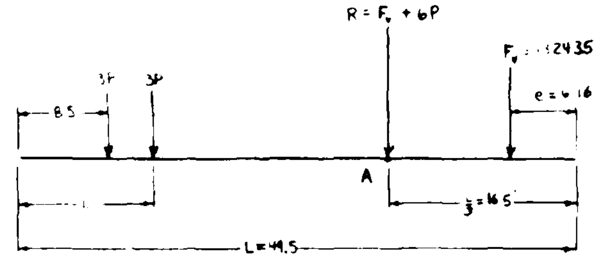


DETAIL A: TYPICAL ANCHORING SYSTEM

Figure A8. (Sheet 2 of 2)

WORKING FORCE, P - NORMAL OPERATION WITH ICE

P = Prestress force per hole required to move resultant (R) to $\frac{1}{3}$ point of base



$\Sigma M_A = 0$

$3P[49.5 - 16.5 - 8.5] + 3P[49.5 - 16.5 - 12.0] = 324.35[16.5 - 6.16]$

$P = 150.3$ Kips

MAXIMUM TEMPORARY POSTTENSIONING FORCE, P_{MAX}

FOR BOND LENGTH 20 ft (Diameter of hole = 4 in)

MAT'L 1 [EL 457 441]

τ_{BOND} = [illegible] psi = 31.10 Ksf (From lab test)

$A_{BOND} = \pi(\frac{4}{12})(18) = 18.85\ ft^2$

MAT'L 2 [EL 441 429]

τ_{BOND} = 31.4 psi = 4.52 Ksf (From lab test)

$A_{BOND} = \pi(\frac{4}{12})(2) = 2.09\ ft^2$

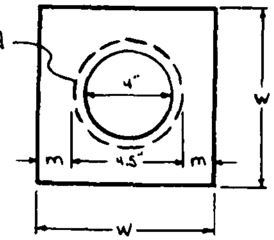
$$\text{SAFETY FACTOR}_{BOND} = \frac{P_{BOND}}{P_{MAX}} = 4.0$$

$$P_{MAX} = \frac{P_{BOND}}{4} = \frac{[31.10(18.85) + 4.52(2.09)]}{4}$$

$$P_{MAX} = 148.9 \text{ Kips}$$

Figure A9. Posttensioning design, tainter gate monolith pier (Sheet 1 of 2)

BEARING PLATE DESIGN



Average compressive strength from lab test of cores taken from monolith = f'_c = 8767 psi

Allowable concrete bearing = F_p = 0.375 f'_c = 3288 psi

Required bearing AREA = $\frac{P_{max}}{F_p}$ = 45.29 IN^2

Required bearing plate width = $\sqrt{45.29 + \pi(2)^2}$ = 7.61 IN, Say 8 inches

F_y = 36,000 psi (A36 Steel)

F_b = 0.75 F_y = 27,000 psi

m = $\frac{8 - 4.5}{2}$ = 1.75 IN

Required plate thickness = $\sqrt{\frac{3 F_p m^2}{F_b}}$ = 1.06 IN

PLATE DIMENSIONS

8" x 8" x 1¼"

Figure A9. (Sheet 2 of 2)

AT INTERFACE (PRECUT CONCRETE-ON-ROCK)

$\phi_i = 30^\circ$, $C_i = 0$, $A = 368\ ft^2$

$R_i = F_V \tan\phi_i + \cancel{C_i A}^{\,0}$

$= 1479.88 \tan 30^\circ = 854.41$ Kips (NORMAL OPERATION)
$= 1324.35 \tan 30^\circ = 764.61$ Kips (NORMAL OPERATION WITH ICE)
$= 1427.58 \tan 30^\circ = 824.21$ Kips (FLOOD CONDITION)
$= 1454.53 \tan 30^\circ = 839.77$ Kips (NORMAL OPERATION WITH EARTHQUAKE)

AT CLAYEY SEAM

$\phi_c = 17^\circ$, $C_c = 5.38$ KSF

$R_c = F_V \tan\phi_c + C_c A$

$= 1479.88 \tan 17^\circ + (5.38)(368) = 2432.28$ Kips (NORMAL OPERATION)
$= 1324.35 \tan 17^\circ + (5.38)(368) = 2384.73$ Kips (NORMAL OPERATION WITH ICE)
$= 1427.58 \tan 17^\circ + (5.38)(368) = 2416.30$ Kips (FLOOD CONDITION)
$= 1454.53 \tan 17^\circ + (5.38)(368) = 2424.53$ Kips (NORMAL OPERATION WITH EARTHQUAKE)

AT OPEN BEDDING PLANE

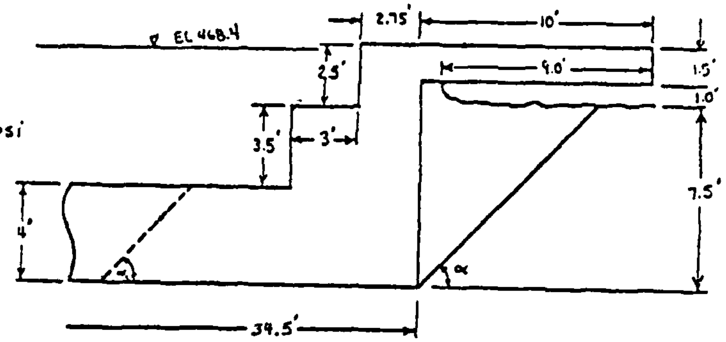
$\phi_o = 25^\circ$, $C_o = 1.30$ KSF

$R_o = F_V \tan\phi_o + C_o A$

$= 1479.88 \tan 25^\circ + (1.3)(368) = 1168.48$ Kips (NORMAL OPERATION)
$= 1324.35 \tan 25^\circ + (1.3)(368) = 1095.95$ Kips (NORMAL OPERATION WITH ICE)
$= 1427.58 \tan 25^\circ + (1.3)(368) = 1144.09$ Kips (FLOOD CONDITION)
$= 1454.53 \tan 25^\circ + (1.3)(368) = 1156.66$ Kips (NORMAL OPERATION WITH EARTHQUAKE)

$R_i < R_c < R_o$ $\therefore$ USE SLIDING RESISTANCE AT INTERFACE

Figure 10. Sliding resistance, tainter gate monolith pier



CONCRETE : SHEAR FAILURE THRU APRON

$\alpha_c = 45°$

S = Shear Strength = $0.1 f'_c$ = $0.1\,[9263]$ = 926.3 psi

$A = \dfrac{zY}{\sin\alpha_c} = \dfrac{4(8)}{\sin 45°} = 45.25\ ft^2$

R'_c = SLIDING RESISTANCE DUE TO SHEAR

$R'_c = S\,A\,\cos\alpha_c = \left[926.3\left(\frac{144}{1000}\right)\right][45.25][\cos 45°]$

R'_c = 4267.93 Kips

FOUNDATION: SHEAR FAILURE THRU FOUNDATION STRUT

$\alpha = 45°$

$\phi = 17°$ (From lab test - intact, clayey foundation sample)

S = Cross-Bedded Shear Strength = 5.26 Ksf (From lab test)

$A = \dfrac{zY}{\sin\alpha} = \dfrac{(7.5)(8)}{\sin 45°} = 84.85$

γ_s = 0.1587 Kcf

$W_{wedge} = (0.1587)\left[\frac{1}{2}(7.5)^2(8)\right]$ = 35.7 Kips

P_p = SLIDING RESISTANCE DUE TO SHEAR

$P_p = W_{wedge}\tan(\phi+\alpha) + \dfrac{S\,A}{\cos\alpha\,[1-\tan\phi\,\tan\alpha]}$

$P_p = (35.7)\tan(17°+45°) + \dfrac{(5.26)(84.85)}{\cos 45°[1-\tan 17°\tan 45°]}$

P_p = 976.27 Kips

SLIDING RESISTANCE OF APRON - NORMAL OPERATION, NORMAL OPERATION WITH ICE AND NORMAL OPERATION WITH EARTHQUAKE

γ_c = 0.1552 Kcf

$W_{APRON} = W_{concrete} + W_{water} - \text{Uplift}$

$W_{APRON} = \left[(0.1552)(8)\left[(4)(28.75)+(3.5)(5.75)+(2.5)(2.75)+(10)(1.5)\right]\right]$
$+\left[(0.0625)(8)\left[(468.4-463)(28.75)+(468.4-466.5)(3)\right]\right]$
$-\left[(0.0625)(8)\left[(468.4-459)(34.5)+(468.4-467.5)(10)\right]\right]$

W_{APRON} = 108.76 Kips

R'' = SLIDING RESISTANCE OF APRON = $W_{APRON}\tan 30°$

R'' = $108.76 \tan 30°$ = 62.79 Kips

Figure A11. Sliding resistance of strut, tainter gate monolith pier (Sheet 1 of 2)

FOUNDATION (CONTINUED)

SLIDING RESISTANCE OF APRON - FLOOD CONDITION

$$W_{APRON} = [(0.1552)(8)[(4)(28.75) + (3.5)(5.75) + (2.5)(2.75) + (10.)(1.5)]]$$
$$+ [(0.0625)(8)[(480.3 - 463)(28.75) + (480.3 - 466.5)(3)]]$$
$$- [(0.0625)(8)[(480.3 - 459)(34.5) + [480.3 - 467.5)(10)]]$$

$W_{APRON} = 32.89$ Kips

$$R'' = W_{APRON} \tan 30° = 32.89 \tan 30° = 18.99 \text{ Kips}$$

TOTAL SLIDING RESISTANCE OF APRON - NORMAL OPERATION, NORMAL OPERATION WITH ICE AND NORMAL OPERATION WITH EARTHQUAKE

$$R'_F = P_p + R'' = 976.27 + 62.79 = 1039.06 \text{ Kips}$$

TOTAL SLIDING RESISTANCE OF APRON - FLOOD CONDITION

$$R'_F = 976.27 + 18.99 = 995.26 \text{ Kips}$$

$R'_C > R'_F$ ∴ USE FOUNDATION STRUT RESISTANCE IN SLIDING RESISTANCE CALCULATIONS (PIER)

Figure A11. (Sheet 2 of 2)

FOUNDATION PRESSURE AT ELEVATION 441 (Interface between foundation materials)

BEFORE POSTTENSIONING

σ_{MAX} = Maximum Intergranular Pressure + Overburden + UPLIFT

$\sigma_{MAX} = 17.92 + 2.86 + 0.59$

$\sigma_{MAX} = 21.37$ Ksf $> \sigma_{ALLOWABLE}$

$\sigma_{ALLOWABLE} = \frac{UC}{4}$

UC = 430 psi (from unconfined compression test)

$\sigma_{ALLOWABLE} = \frac{[430][144]}{4\ [1000]} = 15.48$ ksf

AFTER POSTTENSIONING

$\sigma_{MAX} = 9.73 + 2.86 + 0.59$

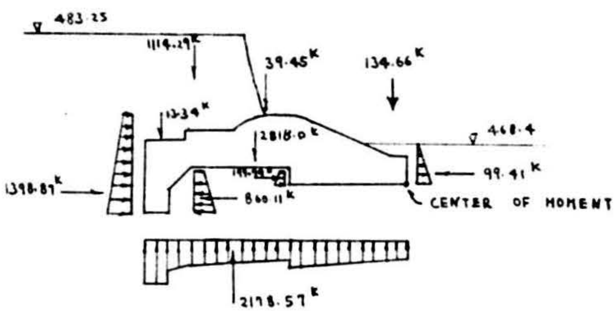
$\sigma_{MAX} = 13.18$ ksf $< \sigma_{ALLOWABLE}$

Figure A12. Bearing pressure at interface between foundation materials, tainter gate monolith pier, normal operation plus ice

STABILITY ANALYSIS OF SPILLWAY ALONE

SIGN CONVENTION

FORCES



ITEM	FACTORS	F_V	F_H	ARM	MOMENT
WT SPWY	[.1552][.275 + .18 + .135 + .035 + .025 + .025 + .1 + .125 + .17 + .23 + 5.76 + 6.30 + 6.66 + 6.93 + 6.95 + 6.90 + 3.87 + 4.6 + 6.70 + 6.36 + 5.90 + 30.47 + 52.5 + 15.13 + 10.73 + 62.56 + 63.0][60]	2818.0		21.53	60671.54
WT HEAD WATER ON SPWY	[.0625][(½)(.5)(1) + (½)(.5)(1) + (½)(.125)(1) + (½)(.0625)(1) + (483.25 − 468)(5.94) + (483.25 − 469)(9.13) + (1)(1.25) + (0.75)(1) + (0.25)(1) + (0.0625)(1) + (5)(483.25 − 470.25)][60]	1114.29		28.8	32091.55
WT TAINTOR ON SPWY		39.45		18.33	723.12
WT SILT	[.0375][(1)(5.93)(1)][60]	13.34		36.33	484.64
$P_{HEAD\ WTR}$ ON SPWY	[.0625][(483.25 − 470.25)(470.25 − 453) + (½)(470.25 − 453)²][60]		−1398.81	1.48	−2070.33
$P_{TAIL\ WTR}$ ON SPWY	[.0625][(468.4 − 459)²(½)][60][.6]		99.41	3.13	311.15
$P_{SPWY\ KEY}$	[(1.44)(4.75) + (.5)(.385)(4.75) + (.953)(5.5) + (½)(.487)(5.5)][60]		860.11	−1.41	−1212.76
$P_{SPWY\ BASE}$	[(.690)(4.25) + (½)(.877 − .69)(4.25)][60]		−199.99	2.04	−407.57
U_{PLIFT} ON SPWY	[(½)(1.89 − 1.825)(3.5) + (1.825)(3.5) + (½)(1.440 − 0.953)(5.5) + (0.953)(5.5) + (½)(0.953 − 0.69)(14.25) + (0.69)(14.25) + (½)(0.877 − 0.586)(15.75) + (0.586)(15.75)][60]	−2178.57		19.80	−43135.69
WT TAIL WTR ON SPWY	[.0625][(468.4 − 463)(2)(1) + (½)(468.4 − 463)(9.3)][60]	134.66		3.87	521.13
		1941.17	−639.14		47,976.78

FIRST TRIAL SOLUTION TO DETERMINE IF PART OF THE BASE IS NOT IN COMPRESSION. IF SO, THEN FULL UPLIFT MUST BE APPLIED UNDER NON - COMPRESSION AREAS

RESULTANT ARM

$$\frac{47,976.78}{1941.17} = 24.72'$$

PERCENT ACTIVE BASE

Figure A13. Stability analysis, tainter gate monolith spillway, normal operation (Sheet 1 of 2)

SLIDING

$R = F_v \tan\phi = 1941.17 \tan 30^\circ = 1120.74$

$\text{FACTOR OF SAFETY} = \dfrac{1120.74}{639.14} = 1.75 > 1.50$ OK

$R' = F_v \tan 67^\circ + \text{SHEAR RESISTANCE IN KEY} + \text{Cohesion}$

$R' = 1941.17 \tan 67^\circ + (3.5)(60)(1.1)\sqrt{9998}\left(\frac{144}{1000}\right) + 2.12(35.5)(60)$

$= 12{,}415 \text{ Kips}$

$\text{FACTOR OF SAFETY} = \dfrac{12{,}415}{639.14} = 19.42 > 4.00$ OK

BASE PRESSURE

INTERGRANULAR PRESSURE

$$\sigma = \frac{F_v}{A} \pm \frac{F_v e c}{I}$$

HEEL $\sigma_{max} = \dfrac{1941.17}{[39][60]} + \dfrac{[1941.17][24.92 - 39/2][39/2]}{60[39]^3/12}$

$= 0.83 + 0.67 = 1.50 \text{ ksf}$

TOE $\sigma_{min} = 0.83 - 0.67 = 0.16 \text{ ksf}$

UPLIFT PRESSURE

HEEL $[483.25 - 459][.0625] = 1.52 \text{ ksf}$

TOE $[468.4 - 459][.0625] = 0.59 \text{ ksf}$

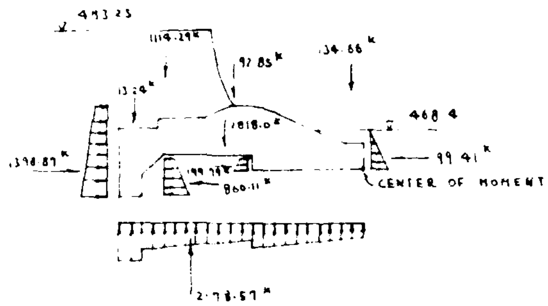
TOTAL PRESSURE = INTERGRANULAR + UPLIFT

HEEL $\sigma_{max} = 1.50 + 1.52 = 3.02 \text{ ksf} < \sigma_{ALLOWABLE} = \dfrac{\text{Unconfined Compressive Strength}}{4} = \dfrac{311}{4} = 77.8 \text{ ksf}$ OK

TOE $\sigma_{min} = 0.16 + 0.59 = 0.75 \text{ ksf}$

Figure A13. (Sheet 2 of 2)

STABILITY ANALYSIS OF SPILLWAY ALONE



ITEM	FACTORS	F_v	F_H	ARM	MOMENT
WT_{SPWY}	[.1552][.275 + .18 + .135 + .035 + .025 + .025 + .1 + .125 + .17 + .23 + 5.76 + 6.30 + 6.66 + 6.93 + 6.95 + 6.90 + 3.87 + 4.6 + 6.70 + 6.36 + 5.90 + 30.47 + 52.5 + 15.13 + 10.73 + 62.56 + 63.0][60]	2818.0		21.53	60671.54
WT HEAD WATER ON SPWY	[.0625][(½)(.5)(1) + (½)(.5)(1) + (½)(.125)(1) + (½)(.0625)(1) + (483.25 − 468)(5.94) + (483.25 − 469)(9.73) + (1)(1.25) + (0.75)(1) + (0.25)(1) + (.0625)(1) + (5)(483.25 − 470.25)][60]	1114.29		28.8	32091.55
$WT_{TAINTOR}$ ON SPWY		92.85		18.33	1701.94
WT_{SILT}	[.0375][(1)(5.93)(1)][60]	13.24		36.33	489.64
$P_{HEAD\ WTR}$ ON SPWY	[.0625][(483.25 − 470.25)(470.25 − 453) + (½)(470.25 − 453)²][60]		−1398.87	1.48	−2070.33
$P_{TAIL\ WTR}$ ON SPWY	[.0625][(468.4 − 459)²(½)][60][.6]		99.41	3.13	311.15
$P_{SPWY\ KEY}$	[(1.44)(4.75) + (½)(.385)(4.75) + (.953)(5.5) + (½)(.487)(5.5)][60]		860.11	−1.41	−1212.76
$P_{SPWY\ BASE}$	[(.690)(4.25) + (1/2)(.877 − .69)(4.25)][60]		−199.79	2.04	−407.57
UPLIFT ON SPWY	[(½)(1.89 − 1.825)(3.5) + (1.825)(3.5) + (½)(1.440 − 0.953)(5.5) + (0.953)(5.5) + (½)(0.953 − 0.69)(14.25) + (0.69)(14.25) + (½)(0.877 − 0.5864)(15.75) + (0.586)(15.75)][60]	−2178.57		19.80	−43135.69
WT TAIL WTR ON SPWY	[.0625][(468.4 − 463)(2)(1) + (½)(468.4 − 463)(9.3)][60]	134.66		3.87	521.13
		1994.47	−639.14		48,955.6

FIRST TRIAL SOLUTION TO DETERMINE IF PART OF THE BASE IS NOT IN COMPRESSION. IF SO, THEN FULL UPLIFT MUST BE APPLIED UNDER NON - COMPRESSION AREAS

RESULTANT ARM

$$\frac{48{,}955.6}{1994.47} = 24.55'$$

Figure A14. Stability analysis, tainter gate monolith spillway, normal operation with ice (Sheet 1 of 2)

PERCENT ACTIVE BASE

$$\frac{(39.0 - 24.55)(3)(100)}{39.0} = 100\ \%$$

SLIDING

$R = F_v \tan\phi = 1994.47 \tan 30° = 1151.51$

$\text{FACTOR OF SAFETY} = \frac{1151.51}{639.14} = 1.80 > 1.50$

BASE PRESSURE

INTERGRANULAR PRESSURE

$$\sigma = \frac{F_v}{A} \pm \frac{F_v e c}{I}$$

HEEL $\sigma_{max} = \frac{1994.47}{(39)(60)} + \frac{(1994.47)(24.55 - 39/2)(39/2)}{60(39)^3/12}$

$= 0.85 + 0.66 = 1.51$ ksf

TOE $\sigma_{min} = 0.85 - 0.66 = 0.19$ ksf

UPLIFT PRESSURE

HEEL $[483.25 - 459][.0625] = 1.52$ ksf

TOE $[468.4 - 459][.0625] = .59$ ksf

TOTAL PRESSURE = INTERGRANULAR + UPLIFT

HEEL $\sigma_{max} = 1.51 + 1.52 = 3.03$ ksf $< \sigma_{ALLOWABLE} = \frac{\text{UNCONFINED COMPRESS STRENGTH}}{4} = \frac{311}{4} = 77.8$ ksf OK

TOE $\sigma_{min} = 0.19 + 0.59 = 0.78$ ksf

$R' = F_v \tan 67° + \text{SHEAR RESISTANCE IN KEY} + \text{COHESION}$

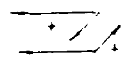
$= 1994.47 \tan 67° + 3.5(60)(1.1)\sqrt{9998}\left(\frac{144}{1000}\right) + 2.12(35.5)(60)$
$= 12{,}540$ Kips

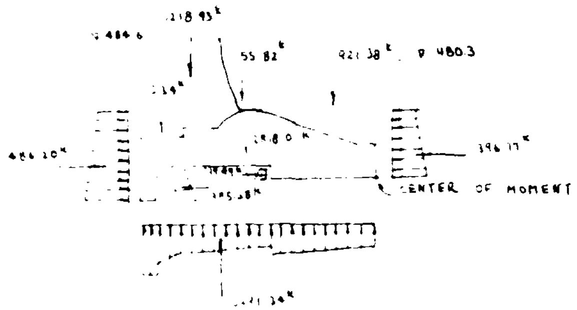
$\text{FACTOR OF SAFETY} = \frac{12{,}540}{639.14} = 19.62 > 4.0$ OK

Figure A14. (Sheet 2 of 2)

STABILITY ANALYSIS OF SPILLWAY ALONE

SIGN CONVENTION





ITEM	FACTORS	F_V	F_H	ARM	MOMENT
WT SPWY	[.1552][.275 + .18 + .135 + .035 + .025 + .025 + .1 + .125 + .19 + .23 + 5.76 + 6.30 + 6.66 + 6.93 + 6.95 + 6.90 + 3.87 + 4.6 + 6.70 + 6.36 + 5.90 + 30.47 + 52.5 + 15.13 + 10.73 + 62.56 + 63.0][60]	2818.0		21.53	60671.54
WT HEAD WTR ON SPWY	NORMAL OPERATION WT + [.0625][1.35][20.67](60)	1218.93		28.8	35105.18
WT TAILWTR ON SPWY	[.0625][.025 + .025 + .1 + .125 + .19 + .23 + (10.05)(1) + (10.1)(1) + (10.15)(1) + (10.35)(1) + (10.6)(1) + (10.94)(1) + (11.4)(10.33) + (½)(5.9)(10.33) + (19.3)(2)][60]	921.38		6.48	5970.64
WT SILT	[.0375][(1)(5.93)(1)][60]	13.34		36.33	484.64
$P_{HEAD\ WTR}$ ON SPWY	[.0625][(484.6 − 490.25)(490.25 − 453) + (½)(490.25 − 453)²][60]		-1486.20	1.55	-2303.61
$P_{SPWY\ KEY}$	[(1.249)(5.5) + (½)(.358)(5.5) + (1.43)(4.75) + (½)(.322)(4.75)][60]		985.38	-1.26	-1241.58
$P_{SPWY\ BASE}$	[(1.173)(4.25) + (½)(.242)(4.25)][60]		-329.91	2.06	-679.74
$P_{TAILWTR}$ ON SPWY	[.0625][(480.3 − 490.25)(490.25 − 459) + (½)(490.25 − 459)²][60][.6]		396.77	4.95	1964.01
UPLIFT ON SPWY	[(1.956)(3.5) + (½)(.019)(3.5) + (1.249)(5.5) + (½)(.385)(5.5) + (1.173)(4.25) + (½)(.0.96)(19.25) + (1.33)(15.75) + (½)(.084)(15.75)][60]	-2221.34		20.20	-63071.07
WT TAINTOR ON SPWY		55.82		18.33	1023.18
		1806.13	-434.02		35923.09

FIRST TRIAL SOLUTION TO DETERMINE IF PART OF THE BASE IS NOT IN COMPRESSION. IF SO, THEN FULL UPLIFT MUST BE APPLIED UNDER NON-COMPRESSION AREAS

RESULTANT ARM

$$\frac{35923.09}{1806.13} = 19.89$$

PERCENT ACTIVE BASE

$$\frac{(39.0 - 19.89)(300)}{39.0} = 100\ \%$$

Figure A15. Stability analysis, tainter gate monolith spillway, flood condition (Sheet 1 of 2)

SLIDING

$R = F_v \tan\phi = 1806.13 \tan 30^\circ = 1042.77$

FACTOR OF SAFETY $= \dfrac{1042.77}{434.02} = 2.40 > 1.50$

$R' = F_v \tan 67^\circ$ + SHEAR RESISTANCE IN KEY + COHESION

$= 1806.13 \tan 67^\circ + 3.5(60)(1.1)\sqrt{9998}\left(\frac{144}{1000}\right) + 2.12(355)(60)$

$= 12{,}097$ Kips

FACTOR OF SAFETY $= \dfrac{12{,}097}{434.02} = 27.87 > 4.00$ OK

BASE PRESSURE

INTERGRANULAR PRESSURE

$\sigma = \dfrac{F_v}{A} \pm \dfrac{F_v e c}{I}$

HEEL $\sigma_{max} = \dfrac{1806.13}{(39)(60)} + \dfrac{1806.13\,[19.89 - 39/2][39/2]}{[60][39]^3/12}$

$= 0.77 + 0.05 = 0.82$ ksf

TOE $\sigma_{min} = 0.77 - 0.05 = 0.72$ ksf

UPLIFT PRESSURE

HEEL $[484.6 - 459][.0625] = 1.60$

TOE $[480.3 - 459][.0625] = 1.33$

TOTAL PRESSURE = INTERGRANULAR + UPLIFT

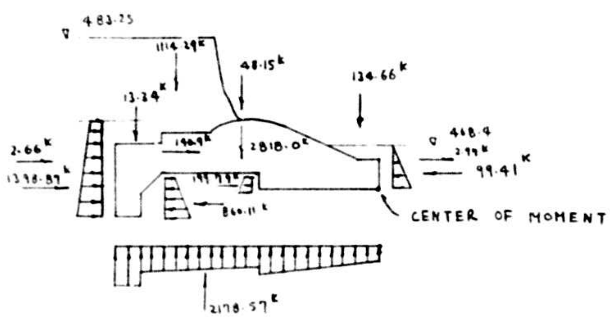
HEEL $\sigma_{max} = 0.82 + 1.60 = 2.42$ ksf $< \sigma_{ALLOWABLE} = \dfrac{\text{UNCONFINED COMPRESS STRENGTH}}{4} = \dfrac{311}{4} = 77.8$ ksf

TOE $\sigma_{min} = 0.72 + 1.33 = 2.05$ ksf

OK

Figure A15. (Sheet 2 of 2)

STABILITY ANALYSIS OF SPILLWAY ALONE



ITEM	FACTORS	F_V	F_H	ARM	MOMENT
WT spwy	[.1552][.275 + .18 + .135 + .035 + .025 + .025 + .1 + .125 + .17 + .23 + 5.76 + 6.30 + 6.66 + 6.93 + 6.95 + 6.90 + 3.87 + 4.6 + 6.70 + 6.36 + 5.90 + 30.47 + 52.5 + 15.13 + 10.75 + 62.56 + 63.0][60]	2818.0		21.53	60671.54
WT HEAD WTR ON SPWY	[.0625][(1/2)(.5)(1) + (1/2)(.5)(1) + (1/2)(.125)(1) + (1/2)(.0625)(1) + (483.25 − 468)(5.94) + (483.25 − 469)(9.73) + (1)(1.25) + (.75)(1) + (.25)(1) + (.0625)(1) + (5)(483.25 − 470.25)][60]	1114.29		28.8	32091.55
WT TAINTOR on SPWY		48.15		18.33	882.59
WT SILT	[.0375][(1)(5.93)(1)][60]	13.34		36.33	484.64
$P_{HEAD\ WTR}$ on SPWY	[.0625][(483.25 − 470.25)(470.25 − 453) + (1/2)(470.25 − 453)^2][60]		−1398.87	1.48	−2070.33
$P_{TAIL\ WTR}$ on SPWY	[.0625][(468.4 − 459)^2 (1/2)][60][.6]		99.41	3.13	311.15
$P_{SPWY\ KEY}$	[(1.44)(4.75) + (1/2)(.385)(4.75) + (.953)(5.5) + (1/2)(.487)(5.5)][60]		860.11	−1.41	−1212.76
$P_{SPWY\ BASE}$	[(.690)(4.25) + (1/2)(.877 − .69)(4.25)][60]		−199.79	2.04	−407.57
UPLIFT on SPWY	[(1/2)(1.89 − 1.825)(3.5) + (1.825)(3.5) + (1/2)(1.440 − 0.953)(5.5) + (.953)(5.5) + (1/2)(.953 − .69)(14.25) + (.69)(14.25) + (1/2)(.877 − .5864)(15.75) + (.586)(15.75)] [60]	−2178.57		19.80	−43135.69
Pe_1	.05 [2817.99]		−140.90	4.52	−636.87
Pe_2 HEAD	(2/3)(51)(.05)(13.0)$\sqrt{(14.25)(13.0)}$ (60) − (2/3)(51)(.05)(14.25)^2(60)		−2.66	.63	−1.68
Pe_2 TAIL	(2/3)(51)(.05)(5.4)$\sqrt{5.4^2}$ (60)		−2.97	6.15	−18.26
WT TAILWTR on SPWY	[.0625][(468.4 − 463)(2)(1) + (1/2)(468.4 − 463)(9.3)][60]	134.66		3.87	521.13
		1949.87	−785.67		47479.44

FIRST TRIAL SOLUTION TO DETERMINE IF PART OF THE BASE IS NOT IN COMPRESSION. IF SO, THEN FULL UPLIFT MUST BE APPLIED UNDER NON-COMPRESSION AREAS.

Figure A16. Stability analysis, tainter gate monolith spillway, normal operation with earthquake (Sheet 1 of 3)

RESULTANT ARM

$\frac{47,479.44}{1949.87} = 24.35$

PERCENT ACTIVE BASE

$\frac{(39.0 - 24.35)(3)(100)}{39.0} = 100\ \%$

SLIDING

$R = F_v \tan\phi = 1949.87 \tan 30^\circ = 1125.76$

$\text{FACTOR OF SAFETY} = \frac{1125.76}{785.67} = 1.43 \geq 1.15$ OK

BASE PRESSURE

INTERGRANULAR PRESSURE

$\sigma = \frac{F_v}{A} \pm \frac{F_v e c}{I}$

HEEL $\sigma_{max} = \frac{1949.87}{(39)(60)} + \frac{1949.87\left[24.35 - \frac{39}{2}\right]\left(\frac{39}{2}\right)}{\frac{60(39)^3}{12}}$

$= 0.83 + 0.62 = 1.45$ ksf

TOE $\sigma_{min} = 0.83 - 0.62 = 0.21$ ksf

UPLIFT PRESSURE

HEEL $[483.25 - 459][.0625] = 1.52$ ksf

TOE $[468.4 - 459][.0625] = 0.59$ ksf

$R' = F_v \tan 69^\circ$ + SHEAR RESISTANCE IN KEY + COHESION

$= 1949.87(2.3559) + (3.5)(60)(1.1)\sqrt{9998}\left(\frac{144}{1000}\right) + 212(35.5)(60)$

$= 12,435$ Kips

$\text{FACTOR OF SAFETY} = \frac{12,435}{785.67} = 15.83 > 4.0$ OK

Figure A16. (Sheet 2 of 3)

TOTAL PRESSURE = INTERGRANULAR + UPLIFT

HEEL $\sigma_{max} = 1.45 + 1.52 = 2.97$ KSF $\angle$ $\sigma_{ALLOWABLE} = \dfrac{\text{UNCONFINED COMPRESS STRENGTH}}{4} = \dfrac{311}{4} = 77.8$ KSF

TOE $\sigma_{min} = 0.21 + 0.59 = 0.8$ KSF

OK

Figure A16. (Sheet 3 of 3)

AT INTERFACE (PRECUT CONCRETE-ON-ROCK)

$\phi_i = 30°$, $C_i = 0$, $A = 2130\ ft^2$

$R_i = F_v \tan\phi_i + \cancel{C_i A}^{\,0}$

$= 1941.17 \tan 30° = 1120.74$ Kips (NORMAL OPERATION)
$= 1994.47 \tan 30° = 1151.51$ Kips (NORMAL OPERATION WITH ICE)
$= 1806.13 \tan 30° = 1042.77$ Kips (FLOOD CONDITION)
$= 1949.87 \tan 30° = 1125.76$ Kips (NORMAL OPERATION WITH EARTHQUAKE)

AT CLAYEY SEAM

$\phi_c = 17°$, $C_c = 5.38$ KSF

$R_c = F_v \tan\phi_c + C_c A$

$= 1941.17 \tan 17° + (5.38)(2130) = 12053.$ Kips (NORMAL OPERATION)
$= 1994.47 \tan 17° + (5.38)(2130) = 12070.$ Kips (NORMAL OPERATION WITH ICE)
$= 1806.13 \tan 17° + (5.38)(2130) = 12012.$ Kips (FLOOD CONDITION)
$= 1949.87 \tan 17° + (5.38)(2130) = 12056.$ Kips (NORMAL OPERATION WITH EARTHQUAKE)

AT OPEN BEDDING PLANE

$\phi_o = 25°$, $C = 1.3$ KSF

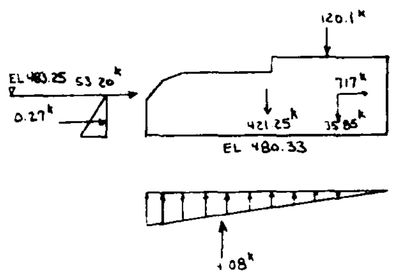
$R_o = F_v \tan\phi_o + C_o A$

$= 1941.17 \tan 25° + (1.3)(2130) = 3674.18$ Kips (NORMAL OPERATION)
$= 1994.47 \tan 25° + (1.3)(2130) = 3699.04$ Kips (NORMAL OPERATION WITH ICE)
$= 1806.13 \tan 25° + (1.3)(2130) = 3611.21$ Kips (FLOOD CONDITION)
$= 1949.87 \tan 25° + (1.3)(2130) = 3678.24$ Kips (NORMAL OPERATION WITH EARTHQUAKE)

$R_i < R_c < R_o$ $\therefore$ USE SLIDING RESISTANCE AT INTERFACE

Figure A17. Sliding resistance, tainter gate monolith spillway

STRESSES @ EL 480.33 DUE TO UNIAXIAL BENDING



RESULTANT ARM

BEFORE POSTTENSIONING

$$e = \frac{4101.23}{573.12} = 7.16'$$

AFTER POSTTENSIONING

$$e = \frac{24,815.18}{1174.92} = 21.12'$$

STRESSES $\left(\sigma = \frac{F_V}{A} \pm \frac{M}{S}\right)$

BEFORE POSTTENSIONING

$$\sigma_{MAX} = \frac{573.12}{(8)(44.67)} + \frac{573.12\,[44.67/2 - 7.16]}{(8)(44.67)^2/6} = 4.87\ ksf = 33.82\ psi$$

$$\sigma_{MIN} = 1.60 - 3.27 = -1.67\ ksf = -11.60\ psi\ (TENSION)$$

ITEM	F_V	F_H	ARM	MOMENT
WT_{PIER}	421.25		15.91	670[illegible].09
P_{ICE}		-53.20	2.92	-155.34
$P_{HEAD\ WATER}$		-0.27	0.97	-0.26
WT_{GATE}	35.85		4.00	143.40
$THRUST_{GATE}$		-717.00	4.92	-3527.64
WT_{BRIDGE}	120.10		8.83	1060.48
UPLIFT	-4.08		29.78	-121.50
SUM BEFORE POSTTENSIONING	573.12	-770.47		4101.23
3P	300.90		36.17	10883.55
3P	300.90		32.67	9830.40
SUM AFTER POSTTENSIONING	1174.92	-770.47		24,815.18

Figure A18. Stresses at elevation 480.33, uniaxial bending, tainter gate monolith pier, normal operation with ice (Sheet 1 of 2)

STRESS @ EL 480.33 DUE TO UNIAXIAL BENDING (CONTINUED)

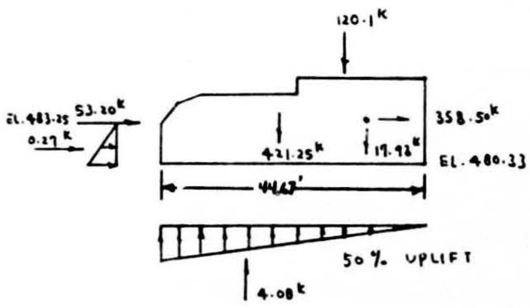
AFTER POSTTENSIONING

$$\sigma_{max} = \frac{1174.92}{(8)(44.67)} + \frac{1174.92\,[44.67/2 - 21.12]}{(8)\,(44.67)^2/6} = 3.83 \text{ ksf} = 26.60 \text{ psi}$$

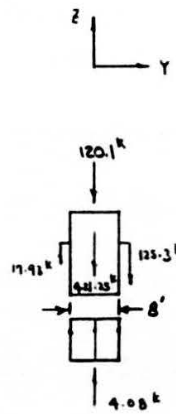
$$\sigma_{min} = 3.29 - 0.54 = 2.75 \text{ ksf} = 19.10 \text{ psi}$$

Figure A18. (Sheet 2 of 2)

STRESSES @ EL. 480.33 DUE TO BIAXIAL BENDING



SIDE VIEW



END VIEW

ITEM	F_V	F_H	ARM	MOMENT M_x	MOMENT M_Y
WT_{PIER}	421.25		15.91	6702.09	
P_{ICE}		-53.20	2.92	-155.34	
$P_{HEAD\ WATER}$		-0.27	0.97	-0.26	
$WT_{GATE\ R}$	125.3		4.00	501.20	
			-4.00		-501.20
$WT_{GATE\ L}$	17.93		4.00	71.72	
			4.00		71.72
$THRUST_{GATE\ L}$		-358.5	4.92	-1763.82	
WT_{BRIDGE}	120.1		8.83	1060.48	
UPLIFT	-4.08		29.78	-121.50	
SUM BEFORE POSTTENSIONING	680.5	-411.97		6294.57	-429.48
3P	300.9		36.17	10883.55	
3P	300.9		32.67	9830.40	
SUM AFTER POSTTENSIONING	1282.3	-411.97		27008.52	-429.48

RESULTANT ARMS

$e_x = \dfrac{6294.57}{680.5} = 9.25'$ (BEFORE POSTTENSIONING)

$e_y = \dfrac{429.48}{680.5} = 0.63'$ (BEFORE POSTTENSIONING)

$e_x = \dfrac{27008.52}{1282.3} = 21.06'$ (AFTER POSTTENSIONING)

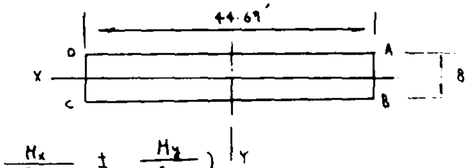
$e_y = \dfrac{429.48}{1282.3} = 0.33'$ (AFTER POSTTENSIONING)

Figure A19. Stresses at elevation 480.33, biaxial bending, tainter gate monolith pier, normal operation with ice (Sheet 1 of 3)

STRESSES @ EL 480.33 DUE TO BIAXIAL BENDING (CONTINUED)

$$S_{xx} = \frac{44.67\,(8)^3}{12(4)} = 476.48 \ ft^3$$

$$S_{YY} = \frac{(8)(44.67)^3}{12(44.67/2)} = 2660.55 \ ft^3$$



STRESSES BEFORE POSTTENSIONING $\left(\sigma = \frac{F_v}{A} \pm \frac{M_x}{S_{yy}} \pm \frac{M_y}{S_{xx}} \right)$

$$\sigma_A = \frac{680.50}{(8)(44.67)} + \frac{(680.5)(44.67/2 - 9.25)}{2660.55} + \frac{(680.5)(0.63)}{476.48}$$

$$= 1.90 + 3.35 + 0.90 = 6.15 \ KSF = 42.71 \ psi$$

$$\sigma_B = 1.90 + 3.35 - 0.90 = 4.35 \ KSF = 30.21 \ psi$$

$$\sigma_C = 1.90 - 3.35 - 0.90 = -2.35 \ KSF = -16.32 \ psi \quad (TENSION)$$

$$\sigma_D = 1.90 - 3.35 + 0.90 = -0.55 \ KSF = -3.82 \ psi \quad (TENSION)$$

Figure A19. (Sheet 2 of 3)

STRESSES AFTER POSTTENSIONING

$$\sigma_A = \frac{1282.3}{(8)(44.67)} + \frac{1282.3\left[\,44.67/2 - 21.06\right]}{2660.55} + \frac{1282.3\,(0.33)}{476.48}$$

$$= 3.59 + 0.61 + 0.89 = 5.09 \text{ KSF} = 35.35 \text{ psi}$$

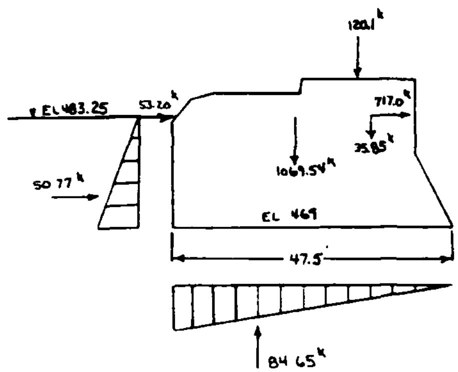
$$\sigma_B = 3.59 + 0.61 - 0.89 = 3.31 \text{ KSF} = 22.99 \text{ psi}$$

$$\sigma_C = 3.59 - 0.61 - 0.89 = 2.09 \text{ KSF} = 14.51 \text{ psi}$$

$$\sigma_D = 3.59 - 0.61 + 0.89 = 3.87 \text{ KSF} = 26.88 \text{ psi}$$

Figure A19. (Sheet 3 of 3)

STRESS @ ELEVATION 469. DUE TO UNIAXIAL BENDING



ITEM	F_V	F_H	ARM	MOMENT
WT_{PIER}	1069.54		21.06	22,524.51
P_{ICE}		-53.20	14.25	-758.10
$P_{HEAD\ WATER}$		-50.77	4.75	-241.16
WT_{GATE}	35.85		6.83	244.86
$THRUST_{GATE}$		-717.00	16.25	-11,651.25
WT_{BRIDGE}	120.10		11.66	1400.37
UPLIFT	-84.65		31.67	-2680.87
SUM BEFORE POSTTENSIONING	1140.84	-820.97		8838.36
3P	300.90		39.0	11,735.10
3P	300.90		35.5	10,681.95
SUM AFTER POSTTENSIONING	1742.64	-820.97		31,255.41

RESULTANT ARM

BEFORE POSTTENSIONING

$$e = \frac{8838.36}{1140.84} = 7.75'$$

AFTER POSTTENSIONING

$$e = \frac{31,255.41}{1742.64} = 17.94'$$

Figure A20. Stresses at elevation 469, uniaxial bending, tainter gate monolith pier, normal operation with ice (Sheet 1 of 2)

STRESSES ② ELEVATION 469, DUE TO UNIAXIAL BENDING $\left(\sigma = \frac{F_v}{A} \pm \frac{M}{S}\right)$

BEFORE POSTTENSIONING

$$\sigma_{MAX} = \frac{1140.84}{(8)(47.5)} + \frac{1140.84\,[47.5/2 - 7.75]}{(8)(47.5)^2/6} = 9.07 \text{ ksf} = 62.99 \text{ psi}$$

$$\sigma_{MIN} = 3.00 - 6.07 = -3.07 \text{ ksf} = -21.32 \text{ psi} \quad \text{(TENSION)}$$

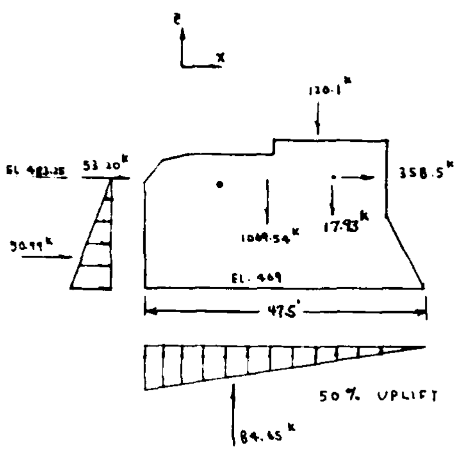
AFTER POSTTENSIONING

$$\sigma_{MAX} = \frac{1742.64}{(8)(47.5)} + \frac{1742.64\,[47.5/2 - 17.94]}{(8)(47.5)^2/6} = 7.96 \text{ ksf} = 55.28 \text{ psi}$$

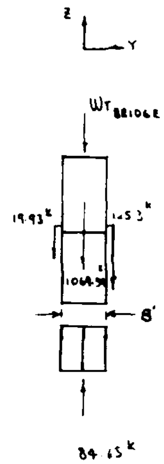
$$\sigma_{MIN} = 4.59 - 3.37 = 1.22 \text{ ksf} = 8.47 \text{ psi}$$

Figure A20. (Sheet 2 of 2)

STRESSES @ EL. 469 DUE TO BIAXIAL BENDING



SIDE VIEW



END VIEW

ITEM	F_V	F_H	ARM	MOMENT M_x	MOMENT M_Y
WT_{PIER}	1069.54		21.06	22,524.51	
P_{ICE}		-53.20	14.2	-758.10	
$P_{HEADWATER}$		-50.77	4.75	-241.16	
WT_{GATE_R}	125.30		6.83	855.80	
			-4.00		-501.2
WT_{GATE_L}	17.93		6.83	122.46	
			4.00		71.92
$THRUST_{GATE_L}$		-358.5	16.25	-5825.63	
WT_{BRIDGE}	120.10		11.66	1400.37	
U_{PLIFT}	-84.65		31.67	-2680.87	
SUM (BEFORE POSTTENSIONING)	1248.22	-462.47		15397.38	-429.48
3P	300.9		39.0	11735.1	
3P	300.9		35.5	10681.95	
SUM (AFTER POSTTENSIONING)	1850.02	-462.47		37814.43	-429.48

RESULTANT ARMS

$$e_x = \frac{15397.38}{1248.22} = 12.34' \text{ (BEFORE POSTTENSIONING)}$$

$$e_y = \frac{429.48}{1248.22} = 0.34' \text{ (BEFORE POSTTENSIONING)}$$

$$e_x = \frac{37814.43}{1850.02} = 20.44' \text{ (AFTER POSTTENSIONING)}$$

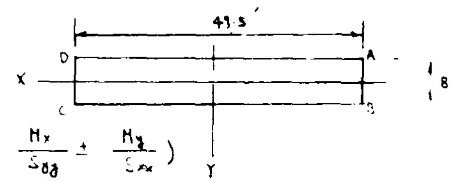
$$e_y = \frac{429.48}{1850.02} = 0.23' \text{ (AFTER POSTTENSIONING)}$$

Figure A21. Stresses at elevation 469, biaxial bending, tainter gate monolith pier, normal operation with ice (Sheet 1 of 3)

STRESSES @ EL 469 DUE TO BIAXIAL BENDING (CONTINUED)

$$S_{xx} = \frac{47.5(8)^3}{12(4)} = 506.67 \ ft^3$$

$$S_{yy} = \frac{8(47.5)^3}{12(47.5/2)} = 3008.33 \ ft^3$$



STRESSES BEFORE POSTTENSIONING $\left(\sigma = \frac{F_v}{A} \pm \frac{M_x}{S_{yy}} \pm \frac{M_y}{S_{xx}} \right)$

$$\sigma_A = \frac{1248.22}{(8)(47.5)} + \frac{(1248.22)(47.5/2 - 12.34)}{2660.55} + \frac{(1248.22)(0.34)}{476.48}$$

$$= 3.28 + 5.35 + 0.89 = 9.52 \ KSF = 66.11 \ PSI$$

$$\sigma_B = 3.28 + 5.35 - 0.89 = 7.74 \ KSF = 53.75 \ PSI$$

$$\sigma_C = 3.28 - 5.35 - 0.89 = -2.96 \ KSF = -20.56 \ PSI \quad (\text{TENSION})$$

$$\sigma_D = 3.28 - 5.35 + 0.89 = -1.18 \ KSF = -8.19 \ PSI \quad (\text{TENSION})$$

Figure A21. (Sheet 2 of 3)

STRESSES @ EL 469 DUE TO BIAXIAL BENDING (CONTINUED)

STRESSES AFTER POSTTENSIONING

$$\sigma_A = \frac{1850.02}{(8)(47.5)} + \frac{(1850.02)(47.56 - 20.44)}{300833} + \frac{1850.02\,(0.23)}{506.67}$$

$$= 4.87 + 2.04 + 0.84 = 7.75 \text{ KSF} = 53.82 \text{ psi}$$

$$\sigma_B = 4.87 + 2.04 - 0.84 = 6.07 \text{ KSF} = 42.15 \text{ psi}$$

$$\sigma_C = 4.87 - 2.04 - 0.84 = 1.99 \text{ KSF} = 13.82 \text{ psi}$$

$$\sigma_D = 4.87 - 2.04 + 0.84 = 3.67 \text{ KSF} = 25.49 \text{ psi}$$

Figure A21. (Sheet 3 of 3)

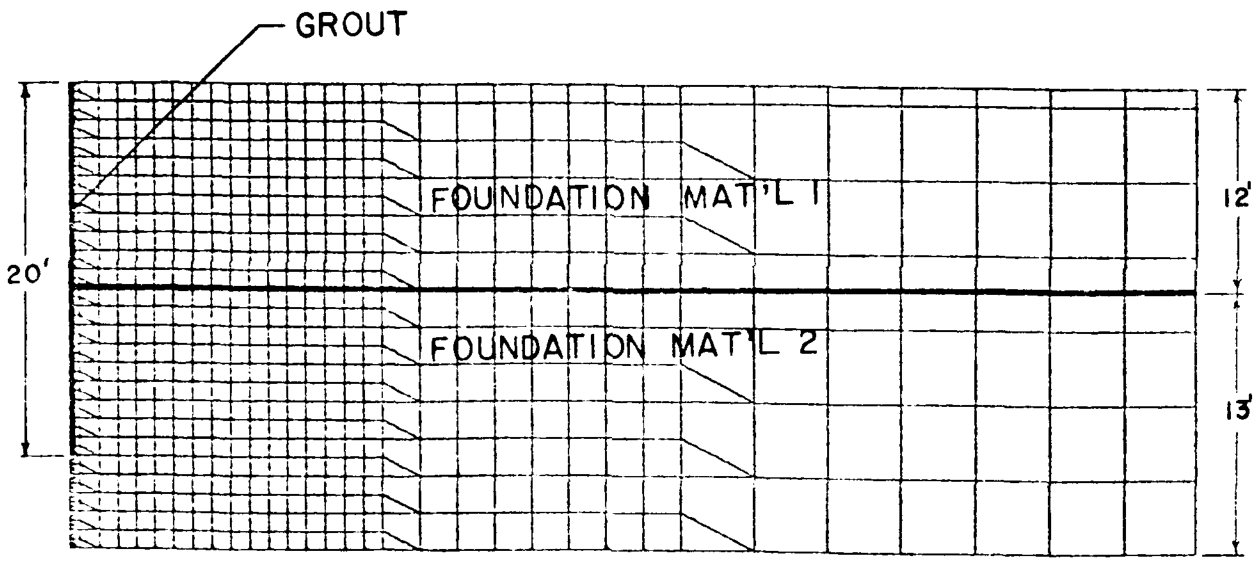


Figure A22. Finite-element grid, tainter gate monolith pier foundation

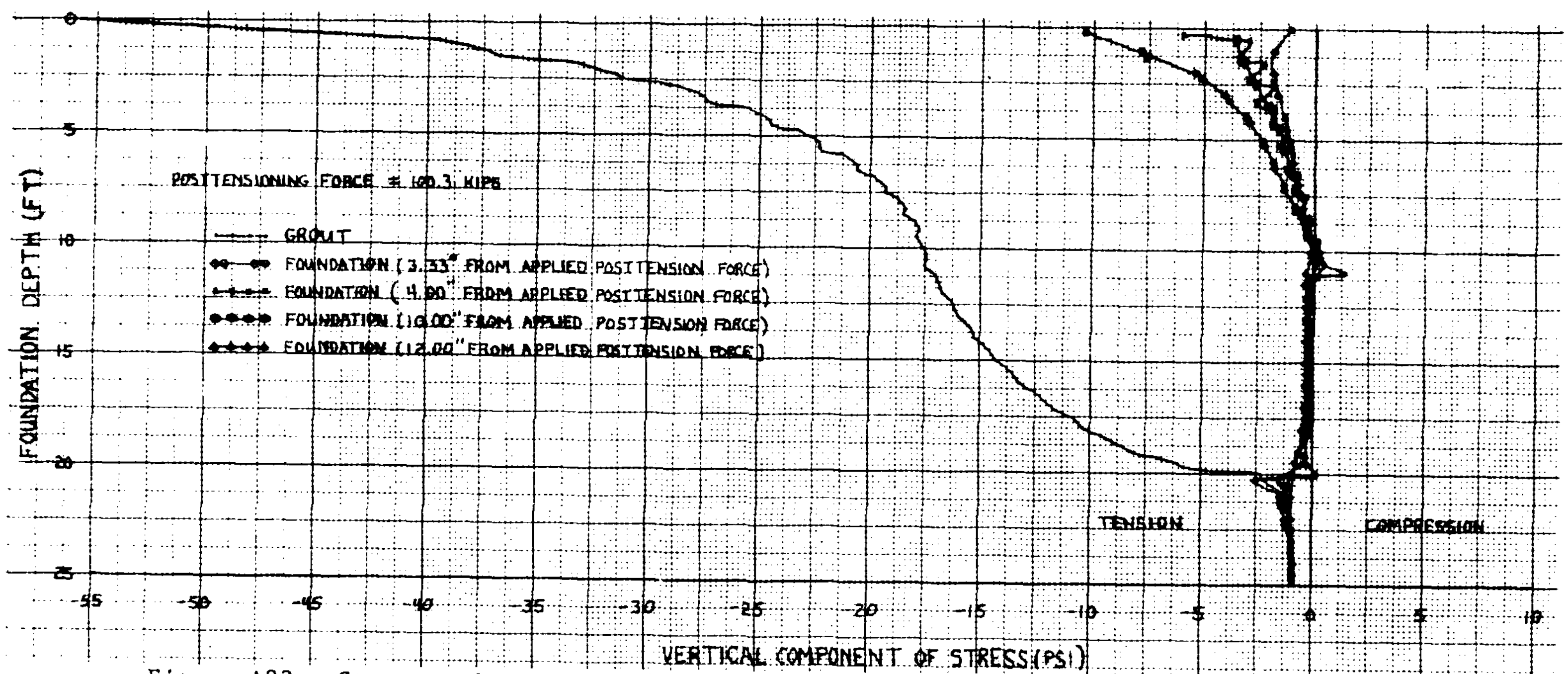


Figure A23. Stresses in grout anchor and foundation due to a 100.3 kip posttensioning force only (other loads not included), tainter gate monolith

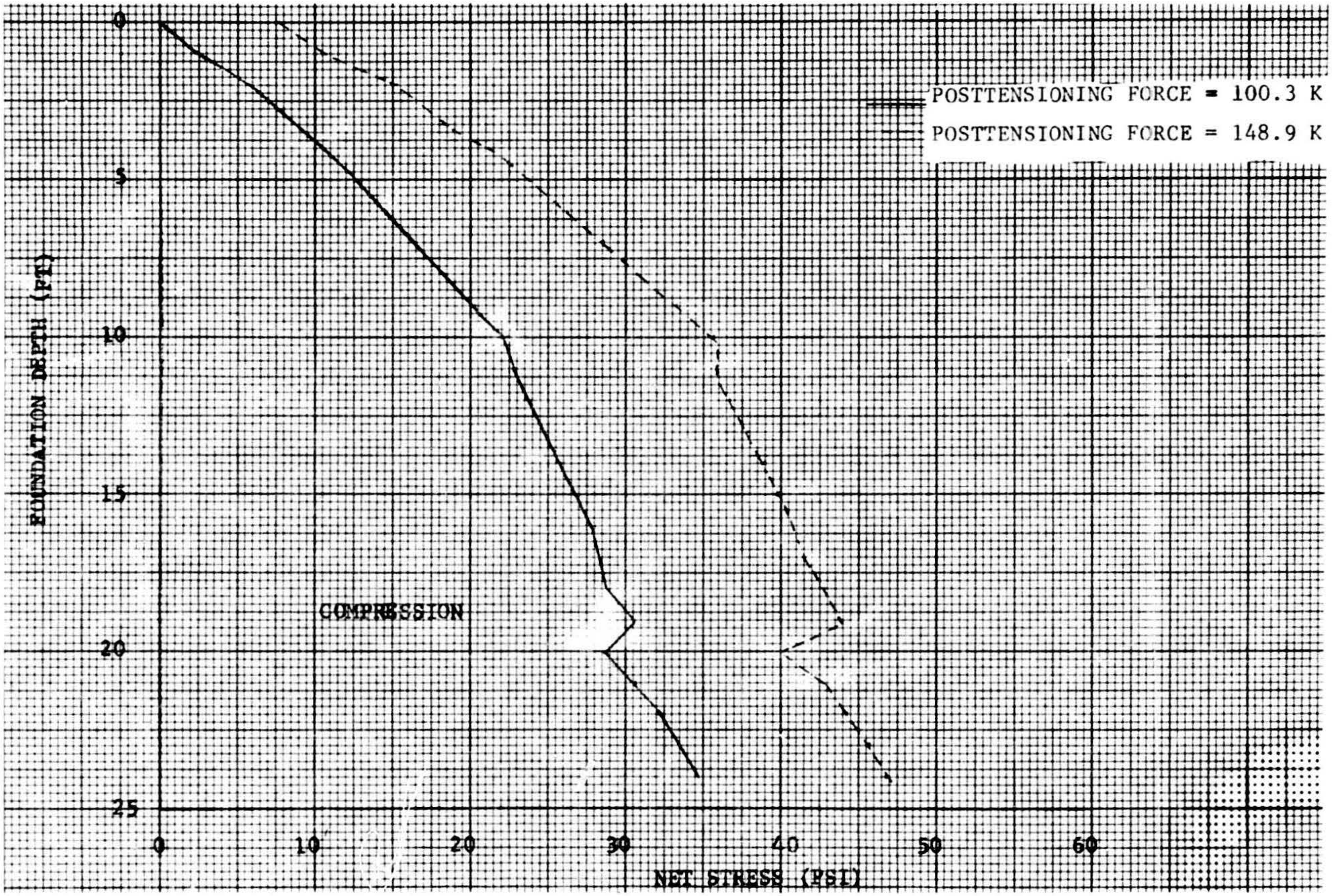


Figure A24. Net foundation stress, tainter gate monolith, normal operation with ice

In accordance with letter from DAEN-RDC, DAEN-ASI dated 22 July 1977, Subject: Facsimile Catalog Cards for Laboratory Technical Publications, a facsimile catalog card in Library of Congress MARC format is reproduced below.

Pace, Carl Eugene
  Stability and stress analyses, Marseilles Dam, Illinois Waterway / by Carl E. Pace, Roy L. Campbell. Vicksburg, Miss. : U. S. Waterways Experiment Station ; Springfield, Va. : available from National Technical Information Service, 1980.
  17, 59 p. : ill. ; 27 cm. (Miscellaneous paper - U. S. Army Engineer Waterways Experiment Station ; SL-80-9)
  Prepared for U. S. Army Engineer District, Chicago, Chicago, Ill.
  References: p. 17.

1. Base pressure. 2. Finite element method. 3. Head gates. 4. Ice chute monolith. 5. Illinois Waterway. 6. Keys (Splines). 7. Marseilles Dam. 8. Monoliths. 9. Post-tensioning. 10. Sluice gates. 11. Stability. 12. Tainter gates. I. Campbell, Roy L., joint author. II. United States. Army. Corps of Engineers. Chicago District. III. Series: United States. Waterways Experiment Station, Vicksburg, Miss. Miscellaneous paper ; SL-80-9.
TA7.W34m no.SL-80-9